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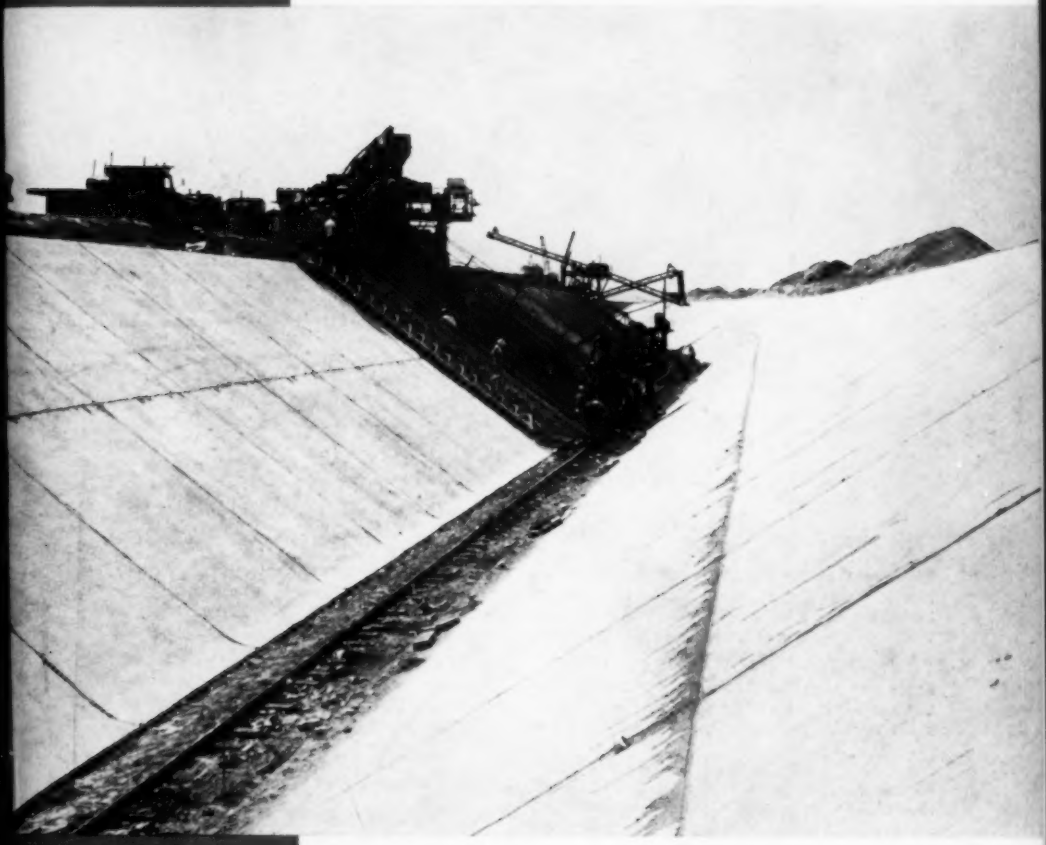

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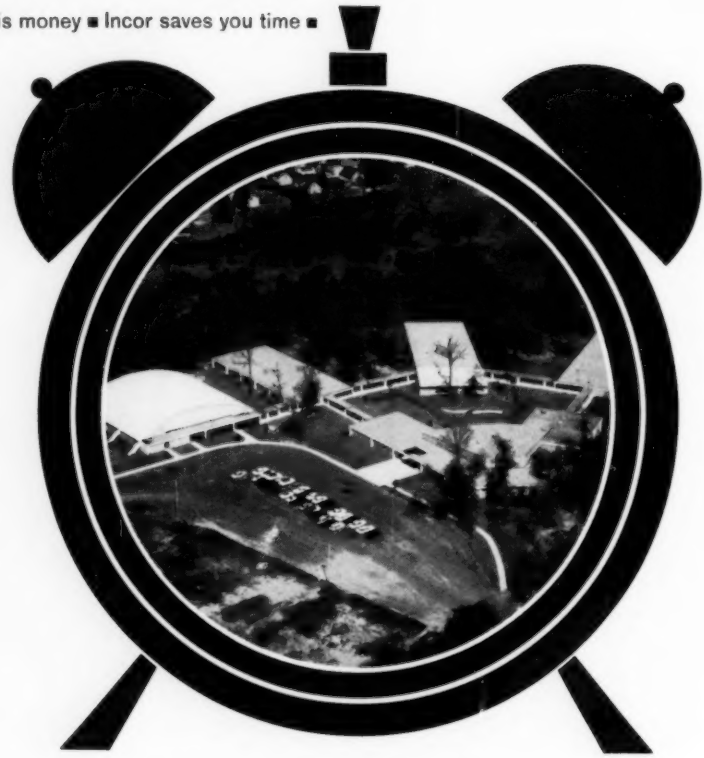

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R. O. HEDSTROM opens the April issue with a report of laboratory "Load Tests of Patterned Concrete Masonry Walls."

"Water-Cement Ratio Versus Strength — Another Look," by H. J. GILKEY focuses attention on the applicability and limitations of the W/C ratio, proposing a modified, duly restricted version of a W/C ratio versus strength relationship.

HARRY H. EDWARDS describes the potential of "Precast and Prestressed Folded Plate Slabs" for roof, floor, and wall construction, and in highway bridges.

JACK R. JANNEY and JOHN F. WISS describe full scale tests on "Load Carrying and Vibration Characteristics of a Multistory Precast Concrete Building."

"Torsional Strength of Prestressed Concrete Members," by PAUL ZIA, reports on results of tests on 68 pretensioned and plain concrete members, T- and I-sections.

WALTER E. RILEY describes a new economical method of thin shell concrete roof construction in his paper, "Shell Construction—A New Approach."

SHU-T'IEN LI presents a general method for determining "Thermal Fixed-End Moments in Columns of Asymmetrical Multispan Integral Frames."

On the Cover—Esquatzel Diversion Canal in the Columbia Basin Irrigation District, Wash. The now completed project measures 64 ft across at the top, 8 ft at the bottom, and is lined with 4½ in. of nonreinforced concrete. Further details appear in the News Letter.—This Earth magazine

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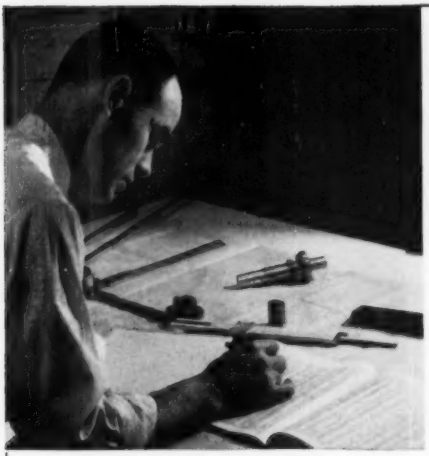

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Part 1 of this report discusses present practice, formwork problems, and the need for improved formwork practice to promote safety, uniformity, and economy in this important phase of concrete construction; Part 2 is a general discussion of responsibility of the engineer or architect and what information (other than that covered by Part 3) should be provided in his plans and specifications; and Part 3 consists of suggested requirements for formwork practice.

In Part 3 Chapters 1, 2, and 3 cover general provisions, minimum design requirements, loads and allowable unit stresses, construction requirements, and form materials applicable to formwork for the great majority of projects involving concrete; Chapters 4 and 5 deal with special concrete structures and special methods of construction. It is believed these suggested formwork practices will be found helpful and suitable for a wide variety of concrete structures.

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PART 1 — PRESENT PRACTICE, PROBLEMS IN FORMWORK, AND NEED FOR GENERAL STANDARDS

■ THIS PART OF THE REPORT is a general discussion of present practice in formwork, and of the need for general criteria useful to engineers, architects, contractors, manufacturers, materials men, and others engaged in or having an interest in concrete formwork.

In 1955, a nationwide questionnaire on present practice was sent to contractors and engineers prominent in concrete construction. Replies were summarized in an interim report of the committee, "Form Construction Practices," ACI JOURNAL, June 1957, *Proceedings* V. 53, pp. 1105-1118.

In only a few cases do contractors have a general policy on form design and construction. Practically none of the replies to the questionnaire indicated that a manual or other written instructions were used. Only a few of the larger well-known contractors engineered their formwork. Construction of formwork today often is haphazard and unstandardized. Furthermore, in many cases forms are overdesigned, causing unnecessarily high costs for those whose methods are overly conservative. There was little agreement as to who should be responsible for construction of formwork—the contractor's superintendent, his foreman (carpentry or concrete), his company engineer, or the owner's representative.

The following quotation illustrates formwork problems encountered even in routine structures.*

> "A system of formwork for the reception of wet concrete at a height above the previously constructed floor may not be the most stable structure. The weight is almost entirely at the top and is supported by an array

*Private communication from Jacob Feld. See also Feld, Jacob, "Failures of Concrete Structures," ACI JOURNAL, Dec. 1957, *Proceedings* V. 54, pp. 449-470.

of posts which may not be rigidly connected to the forms at the top, or to the floor at the bottom.

"Considerable stiffness against lateral sway can be provided if the columns are concreted at least a day ahead of the floor. The unit-labor cost of placing the concrete may be increased, but the small expenditure is cheap insurance. Actually in multistory flat plate designs contractors are convinced that if the columns are cast as soon as the deck is installed, and before any slab reinforcement is placed, a saving will result by avoiding the inconvenience and added cost of placing concrete in columns through the concentrated mass of reinforcing steel at these locations, and in the easier control of bleeding, which reduces finishing time and costs. At the same time, the columns are permitted to take their shrinkage, a procedure usually specified, but often not obeyed."

Neat, well-built, heavily braced forms may still fail due to inadequately tied corners or insufficient provision against uplift. Form failures have occurred when shoring has been inadequately spliced, cross braced, or otherwise improperly engineered to provide for all possible stresses. Posts supported on previously completed floors usually can be assumed to have equal and uniform bearing. However, shores supporting forms for the first floor level about ground are often supported on "mudsills" and may not have uniform bearing. This condition may occur when mudsills rest on soft ground, on backfill recently placed and perhaps softened by surface water, or on frozen ground which may be solid only temporarily until salamanders are fired up. Unequal settlement of mudsills seriously changes post reactions, and may cause serious overloading of posts which do not settle as much as others. To stabilize the soil some contractors use a very lean portland cement mortar tamped in with the soil under the mudsills.

Proper engineering design of formwork often saves contractors more than the price of forms by reducing stripping, removal, and erection time. Formwork is better and more economically constructed on the ground and under the contractor's yard inspection than in the air under field conditions. The average carpenter, though skilled, may find it difficult to conceive of forms as pressure vessels which need to be adequately tied together, braced, and anchored to resist uplift and to be capable of resisting forces in several directions.

If working drawings are made for formwork, the necessary detailed study of structural drawings may uncover errors and omissions. The field work is expedited and the structural engineer is given a chance to see how his design is being interpreted by the contractor. Other results may be shorter job duration, avoidance of field operations getting "hung up" somewhere, more efficient re-use of forms, and better utilization of material and men. There should be "notes to the erector" on such drawings to eliminate need for referring to specific field customs.

ACHIEVING ECONOMY IN FORMWORK

The suggestions which follow are typical of those received from contractors as to how the engineer-architect can plan his work to reduce formwork costs.*

The engineer-architect should consider not only the type of formwork necessary for the quality of construction desired, but also how economy of construction can be achieved. Since cost of formwork may range from 35 to 60 percent of the total cost of a concrete structure, a saving in this item alone can result in significant reductions in the total cost. The following points should be considered and made use of insofar as possible:

1. If columns can be the same size from foundation to roof, beam forms as well as column forms can be re-used from floor to floor without alteration.

2. If spacing of columns is made uniform throughout the building insofar as possible, formwork will be simplified and its mass production made possible.

3. If column centers are set so that the distance *between faces of beams* will be 1 in. plus a multiple of 4 ft, whole sheets of plywood may be used without cutting for slab forms. For instance, if the width of the beam or girder supporting the slab is 14 in., the column centers would be 21 ft 3 in. apart—not 20 ft. In flat slab construction, the drop heads should be 8 ft, 10 ft, or 12 ft square, and the distance between drop heads would be 1 in. plus a multiple of 4 ft. (The extra inch facilitates stripping of the forms.)

4. If columns are the same width as the girders they support, the column form becomes a simple rectangular or square box without cutouts, and the slab form does not have to be cut out at each corner of the column.

5. Ripping of planks can be minimized by making beams all one depth on each floor and of a depth permitting the use of stock lumber, keeping in mind actual rather than nominal lumber dimensions. The width of a beam should be the actual width of a single board, or if necessary, equal to the combined width of two or more boards. If plywood is being used take stock dimensions in detailing structural members. The depth of the beam would thus be the sum of the slab thickness plus $\frac{5}{8}$ in. (thickness of plywood deck) plus actual side board size, minus the thickness of beam bottom board ($1\frac{5}{8}$ in.).

If all beams are made one depth (beams framing into beams as well as beams framing into columns), the whole wooden structure of the forms can be carried on a level table of 4 x 6's supported on shores.

If the engineer-architect takes into consideration stock sizes of lumber and of steel pans in pan-and-joist construction and makes depths of beams and joists the same, savings in labor time in cutting, measuring, and leveling the work will be achieved.

6. Should the structural design be prepared simultaneously with the architectural design, dimensions can be better coordinated. Room sizes can often be varied a few inches to accommodate the structural design.

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*A number of these suggestions are from an article by Joseph R. Farrell, "Economy in Concrete Building," *Civil Engineering*, V. 29, No. 2, Feb. 1959, pp. 44-45.

SAFETY PRECAUTIONS

The following points, most of them having to do with safety, have come up repeatedly in the work of the committee. All of them deserve special attention.

Perhaps those mentioned most often are premature stripping of forms, resulting from a desire for economy, and premature reshoring, reposting, and removal of shores. Carelessness in these operations has caused most of the failures and deficiencies in formwork.

For heavy industrial structures, reshoring seldom is permitted. Stripping and reshoring may cause development of fine, hairline cracks which in later years create a serious maintenance problem. Since proper reshoring is expensive, forms for such structures should be designed so that the original shores are left in place and the forms stripped around them.

For an average multistory industrial building *with proper curing conditions,* three stories of shores with usually be adequate for progress at the rate of one story per week. Shores or reshores should remain in place until a study of the stresses caused by the placing of fresh slabs has been made and compared with design stresses, and until job cured cylinder tests show that no part of the concrete work will be overloaded when any shores or reshores are removed. The 4 x 4-in. shores should not be used as reshores; 6 x 6 in. is the minimum recommended size.

Removal of shores should be approved by the client's representative. His decision should be based not only on the field cured test cylinder results, but also on the history of the recently placed construction. This should include the time and quality of the curing methods as well as accurate information on placing, temperature, and weather conditions. The total load of the upper structure, including freshly placed concrete, formwork, and moving or motor driven equipment thereon, must not exceed the live load for which the lower supporting construction was designed at the concrete strength then available.

Reposting under places in the structure where the design does not provide for the reversal of stress, producing negative moment tension where no tension steel exists, must be prevented.

Inadequate size and spacing of reshores and reposts for emergency loads is a frequent error.

Other deficiencies leading to form failures are:

1. Inadequate cross bracing of shores.

2. Inadequate horizontal and cross bracing, and poor splicing of "double-tier shores" or "multiple-story shores."

3. Failure to control rate of placing concrete vertically without regard to drop in temperatures.

4. Failure to designate rate and order of placing concrete horizontally to prevent unbalanced loadings on the formwork.
5. Unstable soil under mudsills.
6. Failure to inspect formwork during and after concrete placement to detect abnormal deflections or other signs of imminent failure.
7. Lack of proper field inspection by an engineer representing the client. Field inspection by a competent engineer should be required to insure that the design is being properly interpreted.
8. Failure to properly provide for lateral pressures on formwork.
9. Shoring not plumb.
10. Locking devices on metal shoring not locked, inoperative, or missing.
11. Vibration from adjacent moving loads.
12. Exposure to freezing.
13. Overturning by wind.
14. Form damage in excavation by reason of embankment failure.

NEED FOR STANDARDS FOR FORMWORK

Since the cost of the formwork for a concrete structure may be 35 to 60 percent of the total cost of the project, its design and construction demand sound judgment if economy with safety is to result. The engineer or architect responsible for the successful completion of any concrete structure usually will include in his specifications provisions for stripping times, inspection, and approval of formwork procedure which could affect the strength and appearance of the completed structure.

Some contractors feel that the "end result" or performance specification is all that is needed for a contract involving concrete construction. This simple approach may be best when bidding is restricted to highly experienced contractors and for special projects which require competitive formwork solutions. In such special instances specifications must allow freedom to improvise and to apply a choice of equipment and materials.

On the other hand, it is the opinion of the committee that for the vast majority of formwork jobs certain rules and regulations are needed such as minimum design loadings, safe unit design stresses, tolerances, etc. Such general minimum requirements are needed to assure the owner and his engineer or architect that formwork will provide adequate support during the placement of concrete and until it has gained sufficient strength to permit removal of forms. Adequately written and enforced specifications and contractor observance of them should obviate the need for legislation or for provisions in building codes to govern design of formwork, its erection, and its removal.

It is the conclusion of the committee that the design and engineering of the formwork, as well as its construction, must be the responsibility of the contractor. This approach gives him the necessary freedom to use his skill and knowledge to produce an economical finished structure through such means as reusable panels and efficient placement methods.

PART 2 — THE ENGINEER-ARCHITECT SPECIFICATION

GENERAL

For any concrete structure, the specification for formwork written by the engineer or architect will have much to do with the over-all economy and quality of finished work. *Such a specification must be individualized for the particular job, must indicate to the contractor exactly what will be expected from him, and must be so written as to result in economy and safety.*

A well-written formwork specification tends to equalize bids for the work, provided each bidder knows that full compliance will be required of the successful one. Unnecessarily exacting requirements may make bidders question the specification as a whole and may render it virtually impossible for them to be sure just what is expected. They may be overly cautious and overbid or not cautious enough and underbid.

A well-prepared formwork specification is of value not only to the owner and the contractor, but also to the field superintendent of the engineer-architect and to the subcontractors for other trades.

SUGGESTED REQUIREMENTS

Part 3 discusses items requiring attention in preparing the specification for formwork and suggests criteria for assuring proper performance. Some requirements are so written as to allow the contractor discretion where quality of finished concrete work would not be impaired by the use of alternate materials and methods. The engineer-architect may exclude, call special attention to, or strengthen or make more lenient any requirement to best fit the needs of his particular project.

Consideration of applicable provisions of Part 3 in the specification for formwork will not, however, be sufficient to make it complete. To it there must be added requirements for actual materials, finishes, and other items peculiar to and necessary for the individual structure.

Formwork materials and accessories

In his contract plans and specifications, the engineer or architect should specify all formwork materials (such as plywood, dressed and matched lumber, board thicknesses, metal forms, etc.) and may specify type of form ties and other accessories and items if he has reasons to do so. If he does not call for specific materials, accessories, etc., the contractor will be free to use materials of his choice as long as they meet design requirements.

Design, inspection, and approval of formwork

The following items must be clarified in the engineer-architect's specification and drawings:

(a) By whom will formwork be designed?

(b) By whom, when, and for what features will formwork be inspected?

(c) What approvals will be required before concreting, during concreting, and for form removal; and who will make such approvals?

Built-in or attached work, openings or holes through concrete

Plans and specifications of the engineer-architect must cover in detail any requirements for inserts, built-in frames for openings, holes through concrete and similar requirements where work of other trades will be attached to or supported by formwork.

Finish of exposed concrete

Finish requirements for concrete surfaces should be precisely described in *measurable* terms.

Removal of forms

Although the contractor is responsible for design, construction, and safety of formwork, it is recommended that time of removal of forms or shores be approved by the engineer-architect because of the danger of injury to concrete which may not have attained full strength or may be overloaded. Provisions for such approval should be included in the specifications of the engineer-architect.

Concrete control cylinder tests

Suitable tests of job-cured specimens or of concrete in place, methods of evaluating such test results, and minimum standards of strength required should be completely prescribed. The engineer-architect also should specify who will make the cylinders and who will make the tests.

Results of such tests, as well as records of weather conditions and other pertinent information, should be recorded and used by the engineer-architect in deciding when to remove forms and in judging whether or not any part of the concrete work will be injured by their removal.

Tolerances

The suggested tolerances listed in Section 203 of Part 3 are similar to those specified on important work or major structures by many public agencies and private firms. These tolerances or any deviations from them should be specified. It should be remembered that specifying tolerances more exacting than needed may increase construction cost.

SPECIAL PROVISIONS IN ENGINEER-ARCHITECT PLANS AND SPECIFICATIONS

Should the engineer-architect wish formwork drawings submitted for his approval, he should so state. In any case, the contract plans and specifications must include and cover all points necessary to the con-

tractor for his formwork design and in the preparation of his formwork drawings, such as:

(a) Number, location, and details of all construction joints, contraction joints, and expansion joints that will be required for the particular job or parts of it.*

(b) Locations of architectural concrete are to be clearly indicated and/or described in contract drawings and specifications.

Where joints are necessary for architectural treatment and appearance, they must be so indicated on the contract drawings.

(c) Intermediate supports under permanent forms (such as metal deck used for forms, and permanent forms of other materials), supports required by the structural engineer's design for composite action, and any other special supports must be called for in the contract plans and specifications.

(d) The location and order of erection and removal of shores for composite construction must be shown on the engineer's drawings and called for in his specifications.

(e) Special provisions essential for formwork of the special structures and special construction methods (such as shells and folded plates) dealt with in Chapters 4 and 5 of Part 3 must be given on the engineer's contract drawings.

The basic geometry of such structures, as well as the required camber, must be given in sufficient detail on contract drawings and specifications to permit the contractor to construct the form. Where a particular architectural pattern is desired on the formed surface, this pattern must be specified.

(f) Transporation and erection of precast and/or prestressed concrete members are not strictly relevant to a formwork specification; yet they, like formwork, are important factors in the successful and proper placement of concrete members in a concrete structure. Therefore, the engineer-architect's specification may well include the following provisions:

1. *Transportation*—During transportation, precast members must be supported as in their final positions. Deviations may be permitted only when it is shown to the engineer-architect that stability is assured and that no undesirable deflection and cracks will take place.

2. *Erection*—Precast units must be adequately braced and supported during erection to insure proper alignment and safety. Such bracing or support must be maintained until adequate permanent connections have been made.

(g) Should chamfers be required on beam soffits or column corners, they must be specified.

(h) If camber is desired for ceilings or structural members, the contract drawings must so indicate.

The question of allowable camber for precast members, especially prestressed precast members, is an important one. The engineer-architect must specify the amount of camber for any member and the difference in camber between adjacent members which will be acceptable in finished work. Any devices (such as clips to be embedded in edges of members and to be field welded together) desired to reduce or control differential camber should be indicated on the design drawings.

* "Building Code Requirements for Reinforced Concrete" (ACI 318), usually included in contract specifications by reference, has a section (508) on construction joints; also one (503) for items embedded in concrete, such as conduits and pipes.

PART 3 – GENERAL REQUIREMENTS FOR CONCRETE FORMWORK

CHAPTER 1 — DESIGN

101—General

(*a*) Any form regardless of size must be planned in every particular prior to its construction. The thoroughness of planning required will depend upon the size, complexity, and importance (considering re-uses) of the form. In any case, all of the applicable details listed in Section 104 must be included in the planning.

(*b*) Formwork must be adequately designed, erected, supported, braced, and maintained so that it will safely support all vertical and lateral loads that might be applied until such loads can be supported by the concrete structure. Vertical and lateral loads must be carried to the ground by the formwork system or by the completed construction after it has attained adequate strength for that purpose. Formwork must also be so constructed that concrete slabs, walls, and other members will be of correct size in dimensions, shape, alignment, elevation, and position.

102—Loads

(*a*) *Vertical loads*—Vertical loads are assumed to consist of a dead load plus an allowance for live load. The weight of formwork together with the weight of freshly placed concrete is considered dead load. The live load consists of the weight of workmen, equipment, runways, and impact, and will be taken as not less than 50 psf of horizontal projection.

(*b*) *Lateral pressure of concrete*—Forms, ties, and bracing must be designed for a lateral pressure of fresh concrete as follows:

1. For ordinary work with normal internal vibration, in columns,

$$p = 150 + \frac{9000R}{T} \qquad \text{[maximum 3000 psf]}$$

in walls, with rate of placement not exceeding 7 ft per hr

$$p = 150 + \frac{9000R}{T} \qquad \text{[maximum 2000 psf]}$$

and in walls, with rate of placement greater than 7 ft per hr

$$p = 150 + \frac{43400}{T} + \frac{2800R}{T} \qquad \text{[maximum 2000 psf]}$$

Where: p = lateral pressure, psf
R = rate of placement, ft per hr
T = temperature of concrete in the forms, deg F

2. For concrete weighing other than 150 lb per cu ft, revibration, external vibration of forms, cements other than Type I, and/or slumps greater than 4 in., appropriate allowances for change in lateral pressure should be made.*

3. Where retarding admixtures are employed under hot weather conditions an effective value of temperature less than that of the concrete in the forms should be used in the above formula.

When retarding admixtures are used in cold weather the lateral pressure should be assumed as that exerted by a fluid weighing 150 lb per cu ft.

(c) *Lateral loads*—Braces and shores must be designed to resist all foreseeable lateral loads such as wind, cable tensions, inclined supports, dumping of concrete, and starting and stopping of equipment. In no case should the assumed value of lateral load due to wind, dumping of concrete, and equipment acting in any direction at each floor line be less than 100 lb per lineal ft of floor edge or 2 percent of total dead load of the floor, whichever is greater. Wall forms should be designed for a minimum wind load of 10 psf, and bracing for wall forms should be designed for a lateral load of at least 100 lb per lineal ft of wall, applied at the top.

(d) *Special loads*—The formwork must be designed for any special conditions of construction likely to occur, such as unsymmetrical placement of concrete, impact of machine-delivered concrete, uplift, and concentrated loads.

Imposition of construction loads on the partially completed structure should not be allowed except with the approval of the engineer-architect.

103—Design considerations

(a) *Unit stresses*—Unit stresses for use in the design of formwork, exclusive of accessories, are given in the applicable codes or specifications listed in Section 302. When fabricated formwork units are used, manufacturers' recommendations for allowable unit stresses may be followed if supported by test reports or successful experience records.

For forms of a temporary nature with limited re-use, allowable stresses should be those specified in the appropriate design codes or specifications for temporary structures or for temporary loads on permanent structures.

Where there will be a considerable number of form re-uses and where forms to be used many times are fabricated from material such as steel,

*Detailed information on the effect of these special conditions is available in the report "Pressures on Formwork," ACI Committee 622, ACI JOURNAL, Aug. 1958, *Proceedings* V. 55, pp. 173-190, and in the discussion and committee's closure for this report, ACI JOURNAL, June 1959, *Proceedings* V. 55, pp. 1335-1348.

aluminum, or magnesium, it is recommended that the formwork be designed as a permanent structure carrying permanent loads.

In the design of formwork accessories such as form ties, form anchors, and form hangers the following minimum safety factors based on the ultimate strength of the accessory are to be provided except that yield point must not be exceeded. Design capacities guaranteed by reputable manufacturers may be used in lieu of tests for ultimate strength.

Accessory	Safety factor	Type of construction
Form tie	1.5	Light formwork; or ordinary single lifts at grade and 16 ft or less above grade
	2.0	Heavy formwork; all formwork more than 16 ft above grade or unusually hazardous
Form anchor	1.5	Light form panel anchorage only; no hazard to life involved in failure
	2.0	Heavy forms—failure would endanger life—supporting form weight and concrete pressures only
	3.0	Falsework supporting weight of forms, concrete, working loads, and impact
Form hangers	1.5	Light formwork. Design load including total weight of forms and concrete, with 75 psf minimum live load
	2.0	Heavy formwork; form plus concrete weight 100 psf or more; unusually hazardous work

(*b*) *Design*

1. *Analysis*—A design analysis will not be required for formwork for single-story structures less than 15 ft in height if all loads are transmitted directly to the ground by vertical supports. However, safety against buckling of any member should be investigated in all cases. For heavy construction and for all other cases a design analysis is to be made.

2. *Shores*—Shores are defined as vertical or inclined falsework support. When patented shores or methods of shoring are used, manufacturers' recommendations as to load-carrying capacities may be followed but only if supported by test reports from a qualified and recognized testing laboratory; the designer must carefully follow the manufacturer's recommendations as to bracing and working loads for unsupported shore lengths.

Spliced shores must not be used more often than for alternate shores under slabs, or for every third shore under beams, and shores should not be spliced more than once unless diagonal bracing is provided at every splice point. Such spliced shores should be distributed as uniformly as possible throughout the work. To avoid buckling, splices should not be located near the mid-height of the shores nor midway between points of lateral support.

Splices must be designed against buckling and bending as for any other structural compression member. The minimum length of splice

material for timber shores should be 2 ft 4 in. Shores made of round timbers should have three splice pieces at each splice, and those of square timbers should have four splice pieces at each splice.

3. *Diagonal bracing*—The formwork system must be designed to transfer all lateral loads to the ground in such a manner as to insure safety at all times. Diagonal bracing must be provided in vertical and horizontal planes where required to provide stiffness and to prevent buckling of individual members. Where the only bracing requirement is to prevent buckling of individual members, struts may be used if anchored to masonry of adequate strength or to panel points of adjacent braced bays.

4. *Foundations for formwork*—Foundations on ground must consist of properly designed mudsills, spread footings, or pile footings. If soil under mudsills is or may become incapable of supporting superimposed loads without appreciable settlement, it should be stabilized with cement or lean concrete, or by other adequate methods.

5. *Settlement*—Falsework must be so constructed that vertical adjustments can be made to compensate for take-up and settlements, and so that settlements under full load will be a minimum consistent with economy. Where wood timbers are used, the number of horizontal joints and particularly the number of joints where end grain bears upon side grain should be kept at a practical minimum. The unit compressive stress across the grain should not exceed that allowed in Section 103(a). The probable settlement of the falsework, exclusive of foundations, may be approximated as follows: (1) by computing the columnar shortening c in inches, by means of the formula $c = 12SL/E$, in which S is the unit compressive stress per sq in., L is the length of column in ft, and E is the modulus of elasticity; (2) by allowing an assumed value of settlement for each horizontal joint for "taking up" and an additional value at each joint where end grain bears upon side grain for "biting" of the end grain into the side grain. For normal carpentry work each assumed value may be taken as 1/16 in.; where particular care is taken, each may be taken as 1/32 in. The total estimated settlement should approximate the sum of (1) and (2).

Wedges may be used at the top or bottom of shores to facilitate vertical adjustment or dismantling of the formwork. To permit easy removal the maximum load per wedge should be 15 tons when located at the top and 20 tons when located at the bottom.

104—Drawings

(*a*) *General*—Before constructing forms, the contractor, if required, will submit detailed drawings of proposed formwork for examination by the engineer. If such drawings are not satisfactory to the engineer,

the contractor will make such changes as may be required prior to start of work.

All major design values and loading conditions should be shown on formwork drawings. These include assumed values of live load, rate of placement, temperature of concrete, height of drop, weight of moving equipment which may be operated on formwork, foundation pressures, design stresses, camber diagrams, and other pertinent information.

(*b*) *Items included*—In addition to specifying types of materials, sizes, lengths, and connection details, formwork drawings should provide for applicable details such as:

1. Anchors, shores, and braces
2. Field adjustment of the form during placing of concrete
3. Waterstops, keyways, and inserts
4. Working scaffolds and runways
5. Weepholes or vibrator holes where required.
6. Screeds and grade strips
7. Crush plates or wrecking plates where stripping may damage concrete
8. Removal of spreaders or temporary blocking
9. Cleanout holes
10. Construction joints, control joints, and expansion joints to conform to design drawings (ACI Building Code, Section 508)
11. Chamfer strips for exposed corners
12. Camber [See Section 205(a)4]
13. Mudsills or other foundation provisions for formwork
14. Special provisions such as protection from ice and debris at stream crossings, fire, and safety
15. Formwork coatings
16. Notes to formwork erector for conduits, pipes, etc., embedded in concrete according to ACI Building Code, Section 503
17. Details on shoring, reshoring, or leaving original shores in place as forms are stripped

105—Approval by the engineer or architect

Although the safety of formwork is the responsibility of the contractor, the engineer or architect may, under certain circumstances, wish to approve the formwork design. If so, the engineer-architect will include provisions for such approval in his specifications.

Approval might be required for unusually complicated structures, for structures whose designs were predicated upon a particular method of construction, for structures in which the forms impart a desired architectural finish, for folded plates, for thin shells, and for long-span roof structures.

The engineer's approval of the drawings as submitted or as corrected in no way relieves the contractor of his responsibility for adequately constructing and maintaining the forms so that they will function properly.

CHAPTER 2 — CONSTRUCTION

201—General

Construction procedures must be planned in advance to gain proper balance between economy and safety, and should include the design and planning of the formwork.

Forms should be inspected and checked before the reinforcing steel is placed to insure that the concrete will have the dimensions and be in the location shown on the drawings.

Forms must be thoroughly cleaned of all dirt, mortar, and foreign matter before each use. Where the bottom of the form is inaccessible from within, access panels are to be provided to permit thorough removal of extraneous material before placing concrete. Equivalent provisions should be made in forms built of steel or other materials. Forms shall not be re-used after damage from previous use has reached the stage of possible impairment to concrete surfaces if surface appearance is important.

Bulkheads for control joints or construction joints should preferably be made by splitting along the lines of reinforcement passing through the bulkhead so that each portion may be positioned and removed separately. When required on the engineer-architect's drawings, beveled inserts at control joints must be left undisturbed when forms are stripped, and removed only after the concrete has been completely cured and dried out. Wood strips inserted for architectural treatment should be kerfed to permit swelling without pressure on the concrete.

Loading of new slabs should be avoided in the first few days after placement. Loads such as stone, gravel, or other aggregate, timber, boards, reinforcing steel, or support devices, must not be thrown on new construction, nor be allowed to pile up in quantity.

Building materials must not be thrown or piled on the formwork in such manner to damage or overload it.

202—Workmanship

To insure good workmanship the following points warrant careful attention:

(a) Proper splices of studs, wales, or shores

(b) Staggering of joints or splices in sheathing, plywood panels, and bracing

(c) Proper seating of shores

(d) Proper number and location of tie rods or clamps

(e) Proper tightening of tie rods or clamps

(f) Adequate bearing under mudsills; *in no case should mudsills or spread footings rest on frozen ground without written approval of the engineer*

(g) Connection of shores to wales must be adequate to resist uplifts or torsion at joints

(h) Form coatings must be applied before placing of reinforcing steel and must not be used in such quantities as to run onto bars

(i) Details of control joints, construction joints, and expansion joints

203—Suggested tolerances

(*a*) *General*—This section suggests tolerances that are consistent with modern construction practice, considering the effect that permissible deviations will have upon the structural action or operational function of the structure. Surface defects such as "blow holes" and "honeycomb" concrete surfaces are defined as "finishes" and are to be distinguished from tolerances described herein.

Where tolerances are not stated in the specifications or drawings for any individual structure or feature thereof, permissible deviations from established lines, grades, and dimensions are suggested below. The contractor is expected to set and maintain concrete forms *so as to insure completed work within the tolerance limits.*

(*b*) *Tolerances for reinforced concrete buildings*

1. *Variation from the plumb*

a. In the lines and surfaces of columns, piers, walls, and in arrises	In 10 ft	1/4 in.
	In any story or 20 ft maximum	3/8 in.
	In 40 ft or more	3/4 in.
b. For exposed corner columns control-joint grooves, and other conspicuous lines	In any bay or 20 ft maximum	1/4 in.
	In 40 ft or more	1/2 in.

2. *Variation from the level or from the grades indicated on the drawings*

a. In floors, ceilings, beam soffits, and in arrises	In 10 ft	1/4 in.
	In any bay or 20 ft maximum	3/8 in.
	In 40 ft or more	3/4 in.
b. For exposed lintels, sills, parapets, horizontal grooves, and other conspicuous lines	In any bay or 20 ft maximum	1/4 in.
	In 40 ft or more	1/2 in.
3. *Variation of the linear building lines from established position in plan and related position of columns, walls and partitions*	In any bay or 20 ft maximum	1/2 in.
	In 40 ft or more	1 in.
4. *Variation in the sizes and locations of sleeves, floor openings, and wall openings*		1/4 in.

5. *Variation in cross-sectional dimensions of columns and beams and in the thickness of slabs and walls*	Minus Plus	1/4 in. 1/2 in.
6. *Footings*		
a. Variation in dimensions in plan	Minus Plus	1/2 in. 2 in.*
b. Misplacement or eccentricity	2 percent of the footing width in the direction of misplacement but not more than	2 in.*
c. Reduction in thickness	Minus	5 percent of specified thickness
7. *Variation in steps*		
a. In a flight of stairs	Rise Tread	1/8 in. 1/4 in.
b. In consecutive steps	Rise Tread	1/16 in. 1/8 in.

(c) *Tolerances for bridges, viaducts, canal linings, siphons and culverts*

1. *Concrete canal lining*		
a. Departure from established alignment	2 in. on tangents 4 in. on curves	
b. Departure from established profile grade		1 in.
c. Reduction in thickness of lining	10 percent of specified thickness: *provided*, that average thickness is maintained as determined by daily batch volumes	
d. Variation from specified width of section at any height	1/4 of 1 percent plus 1 in.	
e. Variation from established height of lining	1/2 of 1 percent plus 1 in.	
f. Variations in surfaces	Invert 1/4 in. in 10 ft Side slopes 1/2 in. in 10 ft	
2. *Monolithic siphons and culverts*		
a. Departure from established alignment		1 in
b. Departure from established profile grade		1 in

*Applies to concrete only, not to reinforcing bars or dowels.

c. Variation in thickness	At any point: minus 2½ percent or 1/4 in. whichever is greater At any point: plus 5 percent or 1/2 in., whichever is greater	
d. Variation from inside dimensions	1/2 of 1 percent	
e. Variations in surfaces:	Inverts 1/4 in. in 10 ft Side slopes 1/2 in. in 10 ft	

3. *Bridges, checks, overchutes, drops, turnouts, inlets, chutes, and similar structures*

a. Departure from established alignment		1 in.
b. Departure from established grades		1 in.
c. Variation from the plumb or the specified batter in the lines and surfaces of columns, piers, walls, and in arrises	Exposed, in 10 ft Backfilled, in 10 ft	1/2 in. 1 in.
d. Variation from the level or from the grades indicated on the drawings in slabs, beams, horizontal grooves, and railing offsets	Exposed, in 10 ft Backfilled, in 10 ft	1/2 in. 1 in.
e. Variation in cross-sectional dimensions of columns, piers, slabs, walls, beams, and similar parts	Minus Plus	1/4 in. 1/2 in.
f. Variation in thickness of bridge slabs	Minus Plus	1/8 in. 1/4 in.
g. Footings: Same as for footings for buildings [Section 203(b)6]		
h. Variation in the sizes and locations of slab and wall openings		1/2 in.
i. Sills and side walls for radial gates and similar watertight joints. Variation from the plumb or level	Not greater than 1/8 in. in 10 ft	

(d) *Tolerances in mass concrete structures*

1. *All structures*

a. Variation of the constructed linear outline from established position in plan	In 20 ft In 40 ft	1/2 in. 3/4 in.

b. Variations of dimensions to individual structure features from established positions	In 80 ft or more In buried construction	1¼ in. Twice the above amounts
c. Variation from the plumb, from the specified batter, or from the curved surfaces of all structures, including the lines and surfaces of columns, walls, piers, buttresses, arch sections, vertical joint grooves, and visible arrises	In 10 ft In 20 ft In 40 ft or more In buried construction	1/2 in. 3/4 in. 1¼ in. Twice the above amounts
d. Variation from the level or from the grades indicated on the drawings in slabs, beams, soffits, horizontal joint grooves, and visible arrises	In 10 ft In 30 ft or more In buried construction	1/4 in. 1/2 in. Twice the above amounts
e. Variation in cross-sectional dimensions of columns, beams, buttresses, piers, and similar members	Minus Plus	1/4 in. 1/2 in.
f. Variation in the thickness of slabs, walls, arch sections, and similar members	Minus Plus	1/4 in. 1/2 in.
2. *Footings for columns, piers, walls, buttresses, and similar members*		
a. Variation of dimensions in plan	Minus Plus	1/2 in. 2 in.*
b. Misplacement or eccentricity	2 percent of footing width in the direction of misplacement but not more than	2 in.*
c. Reduction in thickness	5 percent of specified thickness	
3. *Sills and side walls for radial gates and similar watertight joints*		
a. Variation from plumb and level	Not greater than 1/8 in. in 10 ft	
(e) *Tolerances for concrete tunnel lining and cast-in-place conduits*		
1. *Departure from established alignment or from established grade*	Free-flow tunnels and conduits High velocity tunnels and conduits Railroad tunnels	1 in. 1/2 in. 1 in.

*Applies to concrete only, not to vertical reinforcing bars or dowels.

2. *Variation in thickness at any point*	Tunnel lining	minus 0
	Conduits	minus 2½ percent or 1/4 in., whichever is greater
	Conduits	plus 5 percent or 1/2 in., whichever is greater
3. *Variation from inside*	1/2 of 1 percent	

(*f*) *Tolerances for pavements*—The following tolerances are suggested for use in the absence of state or local specifications for tolerances for pavements:

1. *Pavements for roadways*	
a. Departure from established alignment	±1/2 in.
b. Departure from established longitudinal grade on any line	±1/4 in.
c. Departure from transverse template contour except at transverse joints	±1/8 in.
d. Departure from transverse template contour at transverse joints	±1/4 in. in width of one traffic lane
2. *Pavements for parking areas, motor pools, repair repair yards, and open storage yards*	Twice values listed for roadway pavements
3. *Pavements for airfield runways, taxiways, and aprons*	
a. Departure from established alignment	±1/4 in.
b. Departure from established finish elevations	±1/2 in.
c. Departure from testing edge of approved 12-ft straight edge:	
1. Runways and taxiways with cross slopes 1 percent or less:	
Longitudinal	1/8 in.
Transverse	3/16 in.
2. Runways and taxiways with cross slopes greater than 1 percent	
Longitudinal	1/8 in.
Transverse	1/4 in.

3. All others with grades 1 percent or less:	
Longitudinal	3/16 in.
Transverse	3/16 in.
4. All others with grades greater than 1 percent:	
Longitudinal	1/4 in.
Transverse	1/4 in.

204—Falsework and centering

(*a*) *Falsework*—Falsework is the temporary structure erected to support work in the process of construction. It is composed of shores or vertical posts, formwork for the beams and/or slabs, and the lateral bracing. The falsework must be supported on satisfactory foundations such as spread footings, mudsills, or piling as provided in Chapter 1.

Shoring resting on intermediate slabs or other construction already in place need not be located directly above shores or reshores below, but the shoring location should be approved by the engineer, unless the construction loads are carried to the ground by the original shores left in place until the construction is completed. All members must be straight and true without twists or bends.

Vertical shores for multifloor scaffolds must be set plumb and in alignment with lower tiers so that loads from upper tiers are transferred directly to the lower tiers. Particular care must also be taken to transfer the lateral loads to the ground or to completed construction of adequate strength [see Section 103(b)3].

Shores or vertical posts must be so erected that they cannot tilt, and in no case should they bear on bricks or stones. Inclined shores must be braced securely against slipping or sliding. The ends of shores should be square cut and have a tight fit at splices. Splices must be secure against bending and buckling as provided in Section 103.

Wedges used for vertical adjustment of falsework should be as indicated in Section 103(b)5.

(*b*) *Centering*—Centering is the highly specialized falsework used in the construction of arches, shells, space structures, or any continuous structure where the entire falsework is lowered (struck or decentered) as a unit to avoid introducing injurious stress in any part of the structure. The lowering of the centering is generally accomplished by the use of sand boxes, jacks, or wedges beneath the supporting members. For the special problems associated with the construction of centering for folded plates, thin shells, and long span roof structures see Chapter 4, Section 404.

(*c*) *Shoring for composite action between previously erected steel or concrete framing and cast-in-place concrete*—See Section 509.

205—Adjustment

(*a*) *Before concreting*

1. Telltale devices should be installed on supported forms and elsewhere as required to facilitate detection and measurement of formwork movements during concreting.

2. Wedges used for final alignment before concrete placement should be secured in position after the final check.

3. Formwork must be so anchored to the shores below that upward or lateral movement of any part of the formwork system will be prevented during concrete placement.

4. To insure that lines and grades of finished concrete work will be within the tolerances recommended in Section 203, the forms must be constructed to the elevation or camber shown on the formwork drawings. These elevations and camber allow for closure of form joints, settlements of mudsills, shrinkage of lumber, dead load deflections, and elastic shortening of form members as well as the camber specified on the structural drawings.

5. Positive means of adjustment (wedges, jacks, sand boxes) should be provided to permit realignment or readjustment of shores if excessive settlement occurs.

6. Runways for moving equipment should be provided with struts or legs as required and should be supported directly on the formwork. They should not bear on or be supported by the reinforcing steel. The formwork must be suitable for the support of such runways without intolerable deflections, vibrations or lateral movements.

(*b*) *During and after concreting*—During and after concreting, the elevations, camber, and plumbness of formwork systems should be checked, using telltale devices. *Appropriate adjustments should be promptly made where necessary.* If, during construction, any weakness develops and the falsework shows any undue settlement or distortion, the work should be stopped, the affected construction removed if permanently damaged, and the falsework strengthened.

Important formwork must be continuously watched so that any corrective measures found necessary may be promptly taken. Formwork watchers must always work under conditions of safety.

206—Removal of forms and supports

Determination of the time of form removal should be based on the effect on the concrete. When forms are stripped there must be no excessive deflection or distortion and no evidence of damage to the concrete,

due either to removal of support or to the stripping operation. Supporting forms and shores must not be removed from beams, floors, and walls until these structural units are strong enough to carry their own weight and any approved superimposed load, which at no time should exceed the live load for which the floor was designed. As a rule the forms for columns and piers may be removed before those for beams and slabs. Forms and scaffolding should be designed so they can be easily and safely removed without impact or shock. When quick re-use of forms is desired, formwork should be designed so that it can be removed without removal of original shores until such time as beam or cylinder tests indicate it is safe to do so.

When field operations are controlled by the engineer's or architect's specifications, the removal of forms, supports and housing, and the discontinuance of heating and curing must follow the requirements of the contract documents. When test beams or cylinders are used to determine stripping times they should be cured under conditions which are not more favorable than the most unfavorable conditions for the portions of the concrete which the test specimens represent. The curing record (time, method, temperature) of the concrete and the weather conditions during the curing of concrete, as well as the test cylinder records, will serve as the basis upon which the engineer will determine his approval of form stripping.

When field operations are not controlled by the specifications, forms and form supports should remain in place for not less than the following periods of time. These periods represent cumulative number of days or fractions thereof, not necessarily consecutive, during which the temperature of the air in contact with the concrete is above 50 F. If high-early-strength cement is used, these periods may be reduced as approved by the engineer.

Arch centers	14 days
Centering under beams	14 days
Floor slabs	7 days
Walls	12-24 hr
Columns	12-24 hr
Sides of beams and all other parts	12-24 hr

207—Reshoring

(*a*) *General*—Reshoring is the construction operation in which the original shores are removed and replaced in a sequence planned to avoid any damage to partially cured concrete. Reshoring is done to facilitate maximum re-use of the formwork and may utilize the strength of the completed construction below. All members must be straight and true without twist or warp. Reshores must be plumb and special care

should be taken in the installation of wedges or jacks to insure concentric loading.

Reshoring is one of the most critical operations in formwork; consequently reshoring procedure must be planned in advance and approved by the engineer. Operations should be performed so that at no time will large areas of new construction be required to support their own weight. *While reshoring is underway, no construction loads should be permitted on the new construction.*

In no case should reshores be so located as to significantly alter the pattern of stress determined in the structural analysis or to induce tensile stresses where reinforcing bars are not provided.

Size and number of reshores must provide a supporting system capable of carrying any loads that may possibly be imposed on it.

Adequate provisions should be made for lateral bracing during this operation. Reshores should be located in the same position on each floor so that they will be continuous in their support from floor to floor, but need not be located directly beneath the new shores above.

(*b*) *Reshoring beam and girder construction*—The forms should be removed from one girder at a time, and the girder should be reshored before any other supports are removed. After all the girders are reshored, the forms should be removed from one beam with its adjacent slabs and the beam should be reshored before any other supports are removed. Each long span slab (clear span 15 ft or more) should be reshored along the center line of the span.

(*c*) *Reshoring flat slabs—The shores for the area within the intersection of the middle strips of each panel should be left in place at all times* during the stripping and reshoring operations. After the other shores in each panel are removed, reshores should be placed at the midpoints between columns and on the column lines before the next panel is stripped.

(*d*) *Removal of reshoring*—Removal of reshoring should be in accordance with Section 206.

CHAPTER 3 — MATERIALS FOR FORMWORK

301—General

The selection of materials suitable for formwork should be based on maximum economy to the contractor, consistent with safety and the quality required in the finished work. Approval by the engineer-architect, if required, should be based only on safety and quality of finished work. These recommendations do not exclude the use of any other material which can be demonstrated to meet these two requirements.

302—Materials

Item	Principal use	Specification	Design data
Steel	Heavy forms and falsework	American Institute of Steel Construction	AISC handbook
	Column and joist forms	Concrete Reinforcing Steel Institute	CRSI
	Permanent forms	American Iron and Steel Institute handbook	AISI handbook section on light gage structural steel members
	Welding of permanent forms	American Welding Society	AISI handbook
Aluminum*	Lightweight panel and framing	Manufacturers' handbook	Manufacturers' handbook
	Permanent forms	Manufacturers' data	Manufacturers' data
Magnesium†	Lightweight panels	Manufacturers' specification	Manufacturers' data
Plywood‡	Form sheathing and panels	U.S. Commercial Standard CS-45-55	Douglas Fir Plywood Association data
Hard board§	Form liner and sheathing	Manufacturers' specification	Manufacturers' specification
	Permanent forms	Manufacturers' data	Manufacturers' data
Lumber	Form framing and sheathing	Grade rules and stress ratings by Southern Pine Association, West Coast Lumbermen's Association, Northeastern Lumbermen's Association, etc.	In United States—National Lumber Manufacturers' Association, "Wood Structural Design Data" In Canada — National Research Council, National Building Code of Canada 1953, Part 4—Design Section 4.3 "Wood"
Fiber or laminated pressed paper tubes or forms	Column forms, voids in concrete slabs, beams, girders, and precast piles	Manufacturers' specifications	Manufacturers' specification and tests. No general design data available

*Shall be readily weldable; nonreactive to concrete; protected again galvanic action at points of contact with steel.

† If the material is affected by standard shearing procedures, an appropriate precaution shall be noted on formwork design drawing.

‡ Also used for similar purposes, western softwood plywood (U.S. Commercial Standard CS-122-56). Design data for these species is also available from Douglas Fir Plywood Association.

§ Check surface reaction with wet concrete.

Item	Principal use	Specification	Design data
Corrugated paper forms	Rectangular or cylindrical voids in beams and slabs; underslab voids in cast-in-place slabs and lift slabs. Normally used with egg crate type internal stiffeners	Manufacturers' specification	Manufacturers' data and tests. No general design data available
Concrete	Footings	ACI Building Code	ACI Building Code
	Permanent forms	ACI Building Code	ACI Building Code
	Precast concrete floor and roof units	ACI 711	ACI Building Code
Fiber glass	Material is not currently in general use for formwork but is used occasionally for special ornamentation	Manufacturers' specification	None available
Form ties, anchors, and hangers	For securing formwork against placing loads and pressures	Manufacturers' specification	Manufacturers' specification and tests. For design values see Section 103
Plaster	Architectural molds	Manufacturers' specification	
Coatings	Easy form removal	Manufacturers' specifications	

303—Accessories*

(*a*) *Form ties*—A form tie is a tensile unit adapted to holding concrete forms against the fluid pressure of freshly placed plastic concrete. In general, it consists of an inside tensile member and an external holding device, both made to specifications of various manufacturers. These

*Most manufacturers furnish information as to the strength of their accessories which, if desired, can be verified by laboratory or field tests.

manufacturers also publish pertinent design data to be used in form design. There are two basic types of tie rods, the prefabricated rod or band type, and the threaded internal disconnecting type. Their capacities range from 1000 to 50,000 lb.

The rod or band type tensile member, with supplemental provision for spreading the forms and a holding device engaging the exterior of the form, is the common type used for light construction [see Section 103(a)].

The threaded internal disconnecting type is more often used for formwork on heavy construction such as heavy foundations, bridges, power houses, locks, dams, and architectural concrete [see Section 103(a)].

Although there is no general uniformity at the present time, a minimum specification for form ties should require that the bearing area of external holding devices be adequate.

(*b*) *Form anchors*—Form anchors are devices used in the securing of formwork to previously placed concrete of adequate strength. The devices normally are embedded in the concrete during placement.

(*c*) *Form hangers*—Form hangers often are used to support form loads from a structural steel framework. Such items must support the dead load of forms, weight of concrete, and construction and impact loads. Form hangers should be symmetrically arranged on the supporting member to minimize twisting or rotation of supporting members.

CHAPTER 4 — SPECIAL STRUCTURES

401—General

In general, formwork for all structures should be designed, constructed and maintained in accordance with recommendations in Chapters 1, 2, and 3. This chapter deals with the additional requirements for formwork for several special classes of work.

402—Architectural concrete

(*a*) *General*—Architectural concrete is any concrete of which one or more surfaces will be permanently exposed to view and the appearance of these surfaces is important from an architectural standpoint. Particular care must be taken in the selection of materials for and in the design and construction of the formwork, as well as in the placing of such concrete, to eliminate bulges, offsets, or other unsightly features in the finished surface.

Location, number, and details of such items as openings, control joints, construction joints, and expansion joints must be shown on the design drawings and form details developed on formwork drawings when they are required [see Section 104(b), Item 10].

(b) *Design*

1. *Lateral pressure of concrete*—Architectural concrete may be subjected to external vibration, revibration, set retardants, and slumps greater than those assumed for determining the lateral pressure as noted in Section 102(b). Particular care must be exercised in these cases to design the forms for the increased lateral pressures arising from the aforementioned sources as noted in Section 102(b). Since deflections in formwork reflect directly in finished surfaces under varying light conditions, forms for architectural concrete must be designed carefully for deflections. Deflections may govern design rather than loads.

2. *Design data for wood forms*—Studs and wales should not be less than 2 x 4 in. S1S1E (surfaced one side and one edge) or S4S (surfaced four sides) except that 3 x 4-in. studs S1S1E or S4S should be used at the vertical joints in plywood sheathing. All studs and wales should be No. 1 or No. 2 common Douglas fir or equal.

All wales should consist of two members, and the joints in the top and bottom members should be staggered at least the spacing of the form ties. Vertical kick strips should be used at the intersection of the wales at all external corners, and the corners should be tightly wedged to prevent leakage.

Under no conditions should studs be spaced at more than 16 in. on centers when used with 1-in. wood sheathing or 11/16 in. and thicker structural grade plywood. Studs should not be spaced more than 12 in. on centers when used with $\frac{5}{8}$-in. plywood. The stud spacing should be reduced 2 in. for all thicknesses of plywood where the grain of the outer piles is parallel to the studs. Wales should not be spaced more than 24 in. on centers and ties should not be spaced more than 27 in. on centers when used with double 2 x 4-in. wales. Ties for use at maximum spacing should have a minimum working strength of 3000 lb when fully assembled.

For wood forms 10 ft or more in height and for all spandrel beams regardless of the height of the forms, double 2 x 6-in. vertical wales, spaced not more than 10 ft on centers and extending the full height of the forms, should be bolted to every other set of horizontal wales to maintain the forms in straight and true alignment.

3. *Approval of proposed form materials and finish*—Sample panels of concrete of prescribed size should be prepared and finished by the contractor for approval by the architect using proposed form materials and form surface treatments, such as wetting, oiling, or lacquers. Such panels must be approved before formwork for architectural concrete is built.

(c) *Construction*

1. *General*—Forms should be substantial, and built to resist the pressures to which they will be subjected. Window and door bucks should be made of 2-in. stock and the wall sheathing should be securely nailed to the bucks with 8d double-headed nails.

2. *Exterior forms—Unlined*

Board sheathing—All forms should be built in place except that prefabricated form sections may be used if a single section will form an entire area from one reveal to another. Panel or patented forms should be used only upon special approval by the architect. The surface finish, grade, or width of boards should be as specified by the architect.

Vertical boards should have all vertical joints plumb, and horizontal boards should have all horizontal joints level and continuous. The vertical joints between horizontal boards should not be over one board wide, should be staggered at least 2 ft, and should be located at studs or girts. Each board must be driven up snug and nailed at every stud or girt with 6d box nails. Six-in. sheathing should be double nailed, and 8-in. and 10-in. boards should be nailed at both edges and at the center. The top three boards below horizontal construction joints should be ripped to approximately one-third of the closure height needed.

Plywood sheathing—The contact surface of all unlined forms should be constructed of ⅝-in. or ¾-in. 5-ply Douglas fir structural plywood of concrete form grade and should conform to U.S. Commercial Standard CS-45-55. All concrete form plywood should be so designated by grade marking on each panel. Full-sized sheets of plywood should be used to insure close fitting, tight joints. All vertical joints should be backed solidly, and the edges of abutting sheets should be nailed to the same stud. Wherever prefabricated form sections are permitted by the architect the joints should be so constructed as to comply with the applicable requirements above.

3. *Exterior forms—Lined*—The backing for form lining should be constructed of a good grade of form lumber that is solid, straight, and free from defects which might impair its strength, but need not be of the quality used for contact forms. Square edged, sized lumber may be used for backing. Form backing should be securely nailed and the edges of the boards should be in contact to prevent any bulging of the lining.

Lining material—Nonwarping fiber board not less than 3/16 in. thick or ¼ in. Douglas fir plywood should be securely nailed to the form sheathing. All lining material should be used in the widest size possible. Areas less than 4 ft wide should be lined with a single width of fiber board or plywood.

Joints in the lining should be offset from those in the backing. The lining material should be nailed to the backing beginning at the center

of the board and working toward the edges to prevent buckling. Nails should have thin flat heads, 2d for ¼-in. plywood and 3d for fiber board, and should be spaced not more than 8 in. on centers at edges of lining material. There should be at least one nail in every square foot of area. A joint approximately 1/16 in. wide should be provided between the edges of adjacent sheets and such joint should be pointed.

4. *Ornamental detail*—Ornamental concrete usually is formed by wood, plastic, or plaster waste molds. Members making up wood molds should be kerfed on the back wherever such members may become wedged between projections in the ornament. Molds must be so constructed that joints will not be opened by slight movement or swelling of the wood. Joints in the molds should be made inconspicuous by pointing.

The molds should be carefully set in the forms and securely held in position to reproduce the design shown on the drawings. Where wood forms adjoin molds, the wood should be neatly fitted to the profile of the mold and all joints should be carefully pointed. The molds and the adjacent wood forms should be so detailed that the wood forms can be stripped without disturbing the molds.

5. *Form ties*—Form ties should be adjustable in length to permit tightening of forms and be of such type as to leave no metal closer than 1½ in. to surface. They should not be fitted with lugs, cones, washers, or other devices which will leave holes through the member larger than ⅞ in. in diameter or depressions larger in diameter than the depth at exposed surface of the concrete. Twisted wire ties should not be permitted. Ties should be tight fitting or pointed to prevent leakage at the holes in the form. Ties that are to be pulled from the wall must be coated with nonstaining bond breaker to facilitate removal. Architectural concrete is classed with heavy formwork in Section 103(a).

Where so specified or indicated on the engineer's drawings, exterior braced forms must be used instead of any of the above mentioned methods to avoid objectionable blemishes in the finished surface.

6. *Forms for control and construction joints (Section 201)*—In architectural concrete, control and/or construction joints must be formed at the junction of the formwork panels, and should be solidly backed to prevent leakage. At joints in forms built in place, a relief strip should be provided and fastened to face of forms.

7. *Removing forms*—Wall forms should not be removed until the concrete has thoroughly hardened, and in no case in less than 4 days unless provisions for equivalent curing are approved by the engineer-architect. Molds must be left in place until they can be removed without damage to the concrete surface.

403—Bridges

(*a*) *General*—For bridges the construction and removal of forms and falsework must be planned in advance. They should be sufficiently rigid to assure that the finished structure will fulfill its intended purpose and will present a pleasing appearance to the public.

(*b*) *Falsework*—See Section 204.

(*c*) *Falsework removal*—Falsework should not be released until the concrete has attained 70 percent of its design strength, and in no case until at least 5 days have elapsed after the concrete has been placed. In continuous structures, falsework should not be released in any span until the first and second adjoining spans on each side has reached the specified strength.

(*d*) *Forms*—Forms may be of wood or metal and must be built mortar tight of sound material sufficiently strong to prevent distortion during placing and curing of the concrete.

(*e*) *Form removal*—Forms for ornamental work, railings, parapets, and vertical surfaces which require a surface finishing operation should be removed not less than 12 hr, nor more than 48 hr after casting the concrete, depending on weather conditions. Bulkheads at construction joints should not be removed for a period of 15 hr after casting adjacent concrete. Forms under slab spans, beams, girders, and brackets must not be removed until the concrete has attained at least 70 percent of its design strength.

(*f*) *Composite construction*—Where composite construction is used, forms and shores, as well as removal of forms and shores, should comply with the contract drawings and specifications as well as Section 509. If required by the design for composite action for both dead and live load, any temporary shores required for this purpose must be called for by the engineer-architect's plans and specifications.

404—Folded plates, thin shells, and long span roof structures

(*a*) *General*—For long span and space structures requiring a complex, three-dimensional design analysis and presenting three-dimensional problems in formwork design, erection and removal, formwork planning should be done by men having the necessary special qualifications and experience. These men should consult and cooperate with the engineer-architect to make sure that the finished surface will conform to his design. The contractor should receive written approval by the engineer-architect of the formwork drawings prior to erection of falsework.

(*b*) *Design*

1. *Lateral loads*—In determining the lateral forces acting on the formwork, the wind load should be a minimum of 15 psf of exposed area in lieu of the 10 psf specified for wall forms in Section 102(c).

2. *Design*—The provisions of Section 103(a) and (b) must be closely adhered to in such formwork planning.

Complete stress analyses should be prepared by competent structural engineers, and the maximum and minimum values of stress, including reversal of stress should be shown for each member. Due regard should be given to unsymmetrical or eccentric loadings which might occur during concrete placement and during erection, decentering, or moving of traveler. The vertical or lateral deflection of the moving forms or travelers as well as the stability under various loads should be investigated to insure that the formwork will function satisfactorily and that the concrete tolerances will be met.

Particular care must be taken in the design and detailing of individual members and connections. Where trussed systems are used, connections must be designed to keep eccentricities as small as possible to minimize deflections or distortions.

Since the weight of the forms and falsework may, in many cases, be equal to or greater than the design live load of the structure, form details should be so designed as to avoid hanging up the form and falsework and thus overloading the structure itself during decentering.

(c) *Drawings*—The contractor should submit detailed drawings of the formwork for approval of the engineer-architect.

These drawings must show the recommended placing sequence and alternate placing sequences of concrete and the resulting loads. To insure that the structure can assume its deflected shape without damage, the decentering or handling procedure of the falsework and forms should be shown on the drawings.

Deflection of these structures often causes binding between the form and the concrete during decentering. Falsework drawings and form details must be planned to prevent binding and to facilitate stripping of forms. Drawings must show such details such as type of inserts and joints in sheathing where spreading of the form may result in the form becoming keyed into the concrete.

(d) *Approval*—The formwork drawings must comply with the contract drawings and specifications and meet the general requirements for formwork to assure the integrity and stability of the structure itself. The engineer should check the design and shop drawings for the formwork to insure that these requirements are met and approve them in writing.

(e) *Removal of forms and falsework*—Forms and falsework should be removed and decentered in the procedure and sequence specified on the shop drawings or on the contract drawings and specifications. Due to the large deflections and the high dead load to live load ratio common to this type of structure, decentering and form removal should not be permitted until tests of job-cured test cylinders demonstrate that the

minimum concrete strength and the modulus of elasticity specified on contract drawings has been reached. Moduli of elasticity may determine time of decentering and required concrete strength at that time may be larger than usual.

405—Mass concrete structures

(*a*) *General*—Generally speaking, mass concrete occurs in heavy civil engineering construction, such as in gravity dams and gravity retaining walls. Special provisions usually are made to control the temperature rise in the mass by the use of cement or cementing material combinations possessing low or moderate heat-generating characteristics.

Formwork for mass concrete falls into two distinct categories, namely, low and high lift. Low lift formwork, for heights 5 to 10 ft, usually consists of multiuse steel cantilever form units which incorporate their own scaffolding and, on occasion, lifting devices. High lift formwork is strictly comparable to the single-use wood forms used extensively for structural concrete.

Because of the large size of mass concrete structures, the rate of placement is low, the depth of each succeeding layer being from 15 to 20 in. With a large working space available behind the form and with little or no obstruction from reinforcing bars, internal vibration is easily accomplished, external vibration of the form is not ordinarily used, and the possibility of mechanical wedging of the aggregate against the form is remote.

(*b*) *Design*

1. *Lateral pressure of concrete*—Forms, ties, tiebacks, and bracing should be designed for a lateral pressure of fresh concrete as given below:

$$p = 120 + \frac{7200R}{T}$$

where: p = lateral pressure, psf
R = rate of placement in ft per hr; generally 1 to 2 ft per hr, but higher for cold weather concreting
T = temperature of concrete in the forms, deg F

For concrete weighing more than 150 lb per cu ft, or more than 4-in. slump, appropriate allowance for additional lateral pressure should be made. Forms adequate for use with Type IV (low heat) cement will be adequate for use with other types of portland cement.

2. *Design considerations*—Particular care must be taken to provide anchorage for forms with a batter and wall forms tied to a rock face. The ultimate strength of the tie rods must not exceed the ultimate strength of the anchor bar or bolt. The bending and welding of high tensile steel tie rods should be prohibited.

(*c*) *Tolerances*—See Section 401.

406—Underground structures

(*a*) *General*—Underground structures comprise many types, such as powerhouses, tunnels, shafts, bomb shelters, factories, and defense installations. They differ from corresponding surface installations in that the construction takes place inside an excavation instead of in the open, thereby providing unique problems in handling and supporting formwork and in the associated concrete placing. As a result, the following four factors usually make the design of formwork for underground structures entirely different than for their above ground counterparts. First, concrete to fill otherwise inaccessible areas may be placed pneumatically or by positive displacement pump and pipeline; second, the excavated rock sometimes is utilized as a form backing thereby permitting the use of rock anchors and tie rods in lieu of external bracing and shores; third, the limits of the excavation demand special handling equipment that adds particular emphasis to the removal and re-use of forms; fourth, rock surfaces sometimes can be used for hoisting devices.

When placement is by pneumatic or positive displacement pump and pipeline methods, the plastic concrete is forced, under pressure, into a void such as the crown of a tunnel lining. This usually is done through a removable pipeline. The end of the pipeline (or "slick" line) usually is embedded between 5 and 10 ft and occasionally up to 20 ft, the line being withdrawn progressively as the cavity fills. With small aggregate and high slump concrete such as might be used in a heavily reinforced tunnel lining, the localized pressures induced by the pump can theoretically reach a maximum of 150 psi less line losses, as the pump stalls or the shear pins fail. When the air gun is used, compressed air at a nominal pressure of 100 psi blasts slugs of high slump concrete into the cavity, and the form must withstand the pressure due to the impact of the concrete and surges of compressed air.

(*b*) *Loads*

1. *Vertical loads*—Vertical and construction loads assumed in design of formwork for underground structures are similar to those for surface structures, with the exception of unusual vertical loads occurring near the crown of arch or tunnel forms and of flotation effect beneath tunnel forms.

Near the crowns of arch and tunnel forms, pressure up to 3000 psf have been induced in areas of overbreak and near vertical bulkheads from concrete placed pneumatically or by positive displacement pump. Until more definite recommendations can be made, the magnitude and distribution of pressure should be determined by the design engineer. In no case should the assumed pressure be less than 1000 psf acting normal to the form plus the dead weight of the concrete placed pneumatically or by pump.

2. *Lateral loads*

(*a*) For shafts and exterior walls against rock the values listed in Section 102(b) should apply.

(*b*) When the shaft form relies on the single shear value of embedded anchors in the previous placement as a means of support, the minimum time lapse between successive placements and maximum allowable loading additional to the dead weight of the form should be specified.

(*c*) For arch forms and for the portions of tunnel forms above the maximum horizontal dimension or spring line of the form, the pressure should be compatible with the pressure discussed under vertical loads in Subsection (b)1 of this section.

(*c*) *Drawings*—In addition to the provisions of Section 401 the following additional data should be included on the drawings for specialized formwork and formwork for tunnels:

1. All pressure diagrams used in the design of the form including diagrams for uplift, for unbalanced lateral or vertical loads, for pressurized concrete, or for any other load applicable to the particular installation.

2. Recommended method of supplemental strutting or bracing to be employed in areas where form pressures may exceed those listed in Subsection (c)1, due to abnormal conditions.

3. Handling diagrams showing the proposed method of handling the form during erection or installation for concrete placement plus the method of bracing, anchorage, etc., during normal operation.

4. In the case of the tunnel arch form, whether it is intended for use with the unit or bulkhead system of concrete placement or is restricted to use with the continuously advancing slope method [see Subsection (d) below].

5. When placement of concrete by pumping or pneumatic methods is anticipated, the capacity and working pressure of the prime mover and the size, length, and maximum embedment of the discharge line should be as assumed in the design. Also, when the design provides for a method of placement other than by sustained pumping via a buried slick line, it should be clearly stated that the design pressures would be exceeded if sustained pumping were adopted.

(*d*) *Construction*—The two basic methods of placing a tunnel arch entail problems in the construction of the formwork that require special provisions to permit proper re-use. These two basic methods are commonly known as the "bulkhead method" and the "continuously advancing slope" method.

The former is used exclusively where poor ground conditions exist so that the lining must be placed concurrently with tunnel driving operations. It is also used when some factor, such as the size of the tunnel, the introduction of reinforcing steel, or the location of construction joints precludes the advancing slope method. The advancing slope method, a continuous method of placement, is usually preferred for

tunnel driven through competent rock, ranging between 10 and 25 ft in diameter and at least 1 mile in length.

The arch form for the bulkhead method usually is fabricated into a single unit between 50 and 150 ft long which is stripped, moved ahead, and re-erected by means of screw jacks or hydraulic rams. These are permanently attached to the form and supporting traveling gantry. The arch form for the continuously advancing slope method usually consists of eight or more sections that range between 15 and 30 ft in length. These are successively stripped or collapsed, telescoped through the other sections and re-erected by means of a form traveler.

Although the minimum stripping time for tunnel arch forms usually is established on the basis of experience, it can be safely predetermined by tests in the laboratory. It is recommended that at the start of a tunnel arch concreting operation, the minimum stripping time be 12 hr for exposed surfaces and 8 hr for construction joints. If the specifications provide for a reduced minimum stripping time based on site experience, such reductions should be in time increments of 30 min or less and should be established by means of laboratory tests and visual inspection and surface scratching of sample areas exposed by opening the form access covers. Arch forms should not be stripped prematurely when unvented ground water seepage could become trapped between the rock surface and the concrete lining.

(*e*) *Materials*—The choice of materials for underground formwork usually is predicted by the shape, degree of re-use and mobility of the form, and the magnitude of pump or pneumatic pressures to which it is subjected. Usually, tunnel and shaft forms are made of steel, or a composite of wood and steel. Experience is of paramount importance in the design and fabrication of a satisfactory tunnel form, due to the nature of the pressures developed by the concrete, placing techniques, and the high degree of mobility usually required.

When re-use is not a factor, plywood and tongue-and-groove lumber sometimes are used for exposed surface finishes, but more consideration may be given to wood sheathing because the high humidity often precludes the normal shrinkage and warping.

CHAPTER 5 — FORMWORK FOR SPECIAL METHODS OF CONSTRUCTION

501—General

The applicable provisions of Chapters 1, 2, and 3 also apply to the work covered in this chapter.

502—Preplaced aggregate concrete

(*a*) *General*—Preplaced-aggregate concrete is made by injecting (intruding) mortar into the voids of a preplaced mass of clean, graded

aggregate. For normal construction the preplaced aggregates are wetted and kept wet until the injection of mortar into the voids is completed. In underwater construction, the mortar displaces the water and fills the voids. In both types of construction this process can create a dense concrete having a high content of coarse aggregate.

The injected mortar contains water, fine sand, portland cement, pozzolanic filler, and an additive designed to increase the penetration and pumpability of the mortar. The special qualities of the additive inhibit early stiffening of the grout, enhance fluidity, and hold the solid constituents in suspension. The coarse aggregate is similar to coarse aggregate for conventional concrete. It is well washed and graded from ½ in. to the largest size practicable. After compaction in the forms, it usually has a void content ranging from 35 to 40 percent.

(*b*) *Design considerations*

1. *Lateral pressure of concrete*—Due to the method of placement, the lateral pressures on formwork are considerably higher than those developed for conventional concrete as given in Section 102(b).

Forms, ties, and bracing should be designed for the sum of:

(a) The lateral pressure of the coarse aggregate as determined from the equivalent fluid lateral pressure of the dry aggregate using the Rankine or Coulomb theories* for granular materials; or a reliable bin action theory; and

(b) The lateral pressure of the injected mortar as an equivalent fluid weighing 130 lb per cu ft. The time required for the initial set of the mortar (from 6 to 24 hr) and the rate of rise (1 to 2 ft per hr) should be ascertained. The maximum height of fluid to be assumed in determining the lateral pressure of the mortar is the product of the rate of rise (ft per hr) and the time of initial set in hours.

The lateral pressure for the design of formwork at any point is the sum of the pressures determined from Step (a) and (b) for the given height.

(*c*) *Construction*—In addition to the provisions of Chapter 2, the forms must be literally "mortar-tight" because preplaced aggregate concrete entails forcing mortar into the voids of the coarse aggregate. The increased lateral pressure usually requires that the workmanship and details of formwork be of better quality than formwork for conventional concrete.

(*d*) *Materials for formwork*—Tongue-and-groove lumber is preferred for exposed surfaces; the joints between boards permit the escape of traces of water. For unexposed surfaces, mortar-tight forms of steel or plywood are acceptable. Prefabricated panel-type forms usually are not suitable because of the difficulty in making mortar-tight seals between panels. Absorptive form linings are not recommended because they permit the coarse aggregate to indent the lining and form an

*Taylor, Donald W., *Fundamentals of Soil Mechanics*, John Wiley and Sons, Inc., New York, 1948.

irregular surface. Form linings, such as hardboard on common sheathing, are not successful because they do not withstand the external form vibration normally required.

503—Slip forms

(*a*) *General*—Placing of concrete by use of slip forms is similar to an extrusion process. Plastic concrete is placed or pumped into the forms, and the forms act as moving dies to shape the concrete. The rate of movement of the forms is regulated so that the forms leave the formed concrete only after it is strong enough to retain its shape while supporting its own weight. Formwork of this type normally is used for vertical structures such as silo and storage bins, bridge piers, shaft type buildings, water tanks, or for horizontal structures such as tunnel inverts, water conduits, drainage channels, canal linings, and paving. Sometimes there may be fixed forms on one side (such as sheathing, rock, earth, or existing masonry) and a sliding form on the other. For other types of work, there are sliding forms on both sides.

Vertical slip forms are moved by jacks which ride on smooth steel rods or pipe embedded in the hardened concrete, whereas horizontal slip forms generally move on a rail system or on a shaped berm. Working decks, concrete supply hoppers, and worker's or finisher's scaffolding, where required, are attached to and carried by the moving formwork.

The vertical or horizontal movement of forms may be a continuous process carried on 24 hr of the day until the structure is completed, or in a planned sequence of finite placement.

This section is divided into two parts: *Vertical slip forms;* and *horizontal slip forms* such as used on drainage channels and canal linings.

Slip forms used on such structures as tunnels and mine shafts should comply with the applicable provisions of Chapter 4, Section 406. Slip forms used on mass concrete structures such as dams should comply with the applicable provisions of Section 405.

(*b*) *Vertical slip forms*

1. *Design considerations*—Slip forms should be designed and constructed and the sliding operation should be carried out under the immediate supervision of a person or persons experienced in slip form work.

In the design of the forms in which jacks on vertical rods are used care must be taken to place jacks in such a manner that the vertical loads are as nearly equal as possible and do not exceed the capacity of the jacks. The steel rods or pipe on which the jacks climb or by which the forms are lifted should be especially designed for the purpose. These rods must be properly braced where not encased in concrete. Jacking rods or pipes may be left in concrete or withdrawn as conditions permit but splices and low bond value must be given special consideration if they are to be used as reinforcement.

The design of the yokes must provide for adequate clearance to install horizontal reinforcing bars in their correct locations prior to their submergence in the rising concrete.

The hydraulic-electric jacking system, which provides for the precise simultaneous movement of the entire form in small preselected increments of approximately 1 in. at 5 to 10 min intervals, is recommended for large structures, especially when single units are involved.

Lateral and diagonal bracing of forms must be provided to insure that the shape of the structure will not be distorted beyond allowable tolerances during the sliding operation.

When slip forms are used for single unit structures in excess of 50 ft diameter, they are usually segmental, or are provided with a substantial center guide made of steel or concrete, to overcome the latent uncertainty of maintaining correct alignment of an otherwise unguided form.

Drawings should be prepared by a competent and experienced engineer employed by the contractor showing the jack layout, formwork, working decks, and scaffolds.

2. *Loads*

a. *Vertical loads*

1. In addition to the dead loads, live loads assumed for design of decks should not be less than the following:

Sheathing and joists	75 psf or concentrated buggy wheel loads whichever is the greater
Beams, trusses, and wales	40 psf

2. Where working decks are used as a bottom form for cast-in-place construction, the deck must be designed for the dead load of the concrete construction plus any superimposed loads, and in no case less than the design loads given in Chapter 1, Section 102. The deflection of the working deck should not exceed ⅛ in. or 1/360 of span, whichever is greater.

3. Vertical loads and possible torsional forces resulting from deck loads and friction of concrete on the forms must also be considered since the forms must act as trusses for the vertical loads between jacks. Knee braces should be provided for top wales where span between jacks exceeds 6 ft or where vertical loads are unusually heavy.

b. *Lateral pressure of concrete*—The lateral pressure of fresh concrete to be used in designing forms, ties, bracing, and wales may be calculated as follows:

$$p = c_1 + \frac{6000R}{T}$$

Where: $c_1 = 100$*
p = lateral pressure, psf
R = rate of concrete placement in ft per hr
T = temperature of concrete in the forms, deg F

Wales must be adequately nailed or bolted together to transmit shear due to lateral pressure of concrete and vertical posts should be placed between wales at lift points.

3. *Construction and materials*—Forms should be between 3 ft 6 in. and 4 ft 6 in. high† and should be constructed of at least 1-in. board, ¾-in. plywood, 10-gage minimum steel sheets, or other approved material. The 1-in. boards should be straight grained and center-matched and placed with the grain running downward and boards spaced 1/16 to ⅛ in. apart to allow for expansion when they become wet. Forms should be erected with slight batter, particularly for the inside faces.

Timber wales should be of 2- or 3-ply lumber at least one ply of which will be 2 in. material. The minimum depth of segmental wales for curved walls should be 4½ in. at the center after cutting.

Special care must be taken in building the forms and arranging the jacks so that the forms will draw straight and true without strain or twist.

4. *Tolerances*—Maximum variation in wall thickness should not exceed ±⅜ in. for walls up to 8 in. thick nor ½ in. for walls thicker than 8 in. The maximum deviation of any point on the slip form with respect to a corresponding reference point at the base of the structure should not exceed 1 in. per 50 ft of height. This is the total deviation which may be composed of translational and rotational components.

5. *Sliding operation*—Maximum rate of slide should be limited by the rate for which the forms are designed. In addition, both maximum and minimum rates of slide must be determined by an experienced slip form supervisor to meet changes in weather, concrete slump, and workability, and the many exigencies which arise during a slide and which cannot be predicted accurately beforehand. A man experienced in slip form construction must be present on the deck at all times during the slide operation.

Forms must be leveled before and after they are filled and must be maintained level throughout the slide. Care must be taken to prevent drifting of the forms from alignment or designed dimensions and to prevent torsional movement.

A water level system using a central drum should be used to maintain the deck level and for placing openings and embedded items; any

*It is felt that $c_1 = 100$ is justified because vibration is slight in slip form work, because concrete is placed in shallow layers of 6 to 10 in., and because there is no revibration.

†The minimum height is a function of the time required for the concrete to gain sufficient strength to support itself without sagging after leaving the slip form. The maximum height provides some working space in the top of the form for placing of concrete and reinforcement. Forms less than 3½ ft high are believed to be dangerously shallow and forms more than 4½ ft high multiply the friction drag unnecessarily.

other equally effective method may be used subject to the approval of the engineer.

Alignment and plumbness of structure should be checked at least once during every 24 hr that the slide is in operation and preferably every 12 to 18 hr.

(c) *Horizontal slip forms for tunnel inverts, drainage channels, and canal linings.**

1. *General*—Linings for tunnel inverts often are constructed in a continuous longitudinally operating method of placement. The transverse section of the invert usually is curved to a prescribed shape. The best way to hold such a shape and at the same time obtain good vibratory consolidation of the higher areas along the side forms is to use a heavy weighted slip form supported on the fixed side forms and having a length equal to or greater than the width of the invert. The slip form is moved forward by winches. The concrete is delivered by means of pump and pipeline or conveyor belt and is placed and vibrated immediately ahead of the slip form.

Linings for drainage channels and canals may be constructed in either a planned sequence of finite placements or a continuous longitudinally operating method of placement. In a planned sequence of finite placements the procedures may range from hand operations for small laterals where the concrete may be dumped and spread on the sides and bottom, to the larger channels where the lining may be placed in alternate sections. In the latter, the bottom slab is placed first to provide support at the toes of the side panels.

A very efficient placement of concrete on slopes is accomplished by use of a weighted, unvibrated steel-faced slip-form screed about 27 in. wide in the direction of movement. The screed may be pulled up the slope by equipment located on the berm or by air hoists mounted on the slip form. The concrete vibrators should be manually operated just ahead of the slip form rather than mounted on the form. If the form is vibrated, this procedure will cause a swell in the finished surface emerging from the trailing edge.

For small channels and canals, a simplified type of slip-form machine has been used with good results. This machine is held to grade and line by a steel pan, shaped to fit the previously prepared excavation section, and is pulled forward by an external source of power. Behind the pan and immediately preceding the slip form is a transverse, compartmented trough for uniformly distributing the mix. Internal vibration of the concrete in the trough may be used to improve the flow of concrete under the slip form, thus eliminating the need for vibration to be applied to the form itself, and also thus decreasing the need for hand finishing.

*The simplified type of slip form is a relatively minor structure; its design is straightforward and is not discussed here. The material in this subsection deals principally with the design and construction of the more elaborate slip-form structures.

This type of form which depends on the subgrade for its support is applicable for placing only unreinforced concrete lining.

For reinforced linings and also for large canal linings, more elaborate slip form machines are required. A framework, traveling on rails on the berm of the drainage channel or canal, supports the working platform, the distributor plate or drop chutes, the compartmented supply trough, vibrator tube in the bottom of the trough, and the slip form. The slip form is a steel plate, curved up at the leading edge, extending across the bottom and up the slopes of the canal and shaped to conform to the finished surface of the lining. When a distributor plate is used, it is fastened to the leading edge of the slip form and extends upward on a steep incline to the working platform. On some of the machines, a continuous row of hoppers in the working platform feed into drop chutes, each supplying one compartment of the trough below. Concrete is dumped, usually from a shuttle car on the working platform, and is guided to the trough below by the distributor plate or the drop chutes.

As the concrete passes out at the bottom of the trough and under the slip form, it is consolidated by a vibrating tube parallel to and a few inches ahead of the leading edge of the form. Consolidation must be accomplished as the concrete passes under the slip form. Proper consolidation cannot be obtained by vibrating the slip form of a lining machine, apparently due to lack of means to supply additional concrete needed to fill the voids. The trailing edge of the slip form is usually adjustable to positions somewhat lower than that of the leading edge. This improves consolidation and tends to mold the concrete more closely to the subgrade. Too low a setting of the trailing edge causes tearing, rather than smoothing, of the surface. On some machines, the slip form is followed within a few feet by an "ironer" plate 18 to 20 in. wide, which, under favorable conditions, leaves a surface that requires little or no hand treatment.

2. *Design considerations*—This specialized formwork should be designed by experienced, competent structural engineers employed by the contractor. A complete structural analysis, including stress diagrams of the machine must be made to insure satisfactory performance. Due regard should be given to unsymmetrical and eccentric loadings and the fact that the machine must be regularly disassembled and assembled as it encounters siphons, bridges, chutes, etc., along the waterway. The vertical or lateral deflections, particularly of long span machines, must be investigated, and sufficient rigidity provided to insure that concrete tolerances will be met. The stability of the machine under the aforementioned loading conditions must be carefully investigated to insure satisfactory performance.

3. *Drawings*—The general provisions of "Drawings" in Section 104 should be met and the contractor should submit drawings of the slip form for review and approval by the engineer. These drawings should

show the handling diagrams, the placing procedure, and the provisions for insuring the required concrete surfaces.

504—Lift method of construction

(*a*) *General*—The lift method of construction consists basically of casting reinforced concrete floor and roof slabs one on top of the other at or near ground level and, after proper curing, of lifting the slabs to their final positions by the use of jacks.

In this "layer" method of precasting, the slabs are commonly of the flat soffit type. Other uniform depth types of framing such as the waffle, joist, and thin shell construction can also be used, with or without prestressing. Top slabs only sometimes utilize upturned beam framing. Patented methods, equipment, and materials for the lifting operations are available.

(*b*) *Design*—If required, special provisions should be made for the ground floor slab and/or the soil supporting it to carry the dead load of the stacked slabs. (*The engineer or architect responsible for design must cover these special provisions in his plans and specifications.*)

(*c*) *Construction*—Care must be taken to assure that collar keyholes or other lifting attachment openings are in direct vertical alignment for all slabs. Wedges should be inserted between the collar opening and column to maintain clearance on all sides of the column. Blockouts should be provided to prevent concrete from entering space between collar and column as well as the lifting attachment openings.

Sleeves, conduits, thimbles, and other similar items should be accurately placed and well secured prior to placing of concrete. Turn-up extensions above the floor line should be sleeved or wrapped with suitable material before casting the next slab above to prevent adhesion to that slab.

Slabs should be lifted only after the concrete has reached a minimum specified strength as determined by field-cured test cylinders and job history of concrete curing. Such minimum strength should be specified by the engineer and should be that required for the slab to carry the dead load and any superimposed loads.

Lifting of all parts of the slab must be simultaneous and at a uniform rate. As each slab is lifted, it should be maintained within the lesser value of 1/360 of span or 1 in. maximum of a theoretical plane throughout all of its lifting points. The lifting equipment must be constantly engaged to prevent slippage or retrogression of the slab during lifting operations.

Construction loads may be imposed upon the slabs during lifting and before final connection to columns only upon approval of the engineer.

(*d*) *Materials*—The material for collars must conform to the contract specifications and drawings.

Bond between stacked slabs can be prevented by the application of a separation compound to all slabs used as casting bases. Wax-free compounds should be used on those slabs that require a separate finish,

tile adhesives, or paint as a later application. Manufacturer's specifications for the application and coverage of the separating media should be followed.

(*e*) *Special conditions*—Shop drawings and a detailed description of the entire lifting operation should be submitted for the engineer's approval. In addition to the applicable items required in Section 104, the shop drawings and description should also include the following:

1. Provisions for carrying the dead load of the stacked slabs
2. Type of form to be used if other than flat soffit slabs
3. Closure strip locations, widths, forming and shoring details
4. Any construction loads to be superimposed upon the slabs during and after lifting operations
5. Description of separating media
6. Provisions for holding in place all forms and embedded items
7. Detailed description of the lifting sequence, including rate of lift, number of slabs and heights to be lifted, provisions for bracing and plumbing the columns

505—Permanent forms

(*a*) *General*—Permanent forms, as the name implies, are forms left in place that may or may not become an integral part of the structure. These forms may be the rigid type such as metal deck, precast concrete, wood, plastics, and various types of fiberboard; or the flexible type such as reinforced water-repellent corrugated paper, or wire mesh with waterproof paper backing.

Where the permanent form is used as a deck form it is generally supported from the main structural frame with or without an intermediate system of temporary supports.

(*b*) *Design considerations*—If the permanent type form is not covered in the engineer's specifications, (1) the manufacturer's specifications should be used; (2) the manufacturer's recommended practice* should be followed for size, span, fastenings, and other special features pertinent to this type of form, such as being water repellent and protected against chemical attack from wet concrete; and (3) the minimum requirements of Chapters 1 and 2 should be adhered to. Particular care should be taken in the design of such forms to minimize distortion or deformation of the form or supporting members under the construction loads.

Where metal deck to become an integral part of the structure is used as a permanent form, its form, depth, gage, physical dimensions, and properties should be as called for on contract drawings and specifications.

(*c*) *Installation*

1. *Shop drawings*—The contractor should submit fully detailed shop drawings for all permanent deck form to the engineer for approval. Shop drawings should show all form thicknesses, metal gages, physical

*If supported by tests by a recognized commercial testing laboratory.

dimensions and properties, accessories, finishes, and methods of attachment to the various classes of the work.

2. *Fastenings*—The permanent deck form must be properly fastened to supporting members and to adjacent sections of form, and properly lapped to provide a tight joint that will prevent loss of mortar during the placement of concrete. End closures for corrugated or fluted forms should be provided where required together with fill pieces around columns, openings, or other places where a tight fit is required.

Flexible types of forms (those that depend for lateral stiffness on supporting members) must be drawn tight for proper installation. Adequate temporary bracing or anchors must be provided in the plane of the top chord of the supporting members to prevent lateral buckling and rotation of these supports and to maintain the required tension in the flexible form.

Paper or metal forms used to form voids in concrete construction should be properly placed and anchored to reinforcement and to side or deck forms with wire ties or by other approved methods to prevent displacement or flotation during placing of concrete. End closures should be properly vented where necessary to minimize cracking of concrete by reason of expansion of air in voids due to the heat of hydration of the concrete.

3. *Electrical raceways*—Where permanent deck forms are used as electrical raceways, units must be installed in conformance with the requirements of the National Electrical Code. Butted ends must be square and smooth on interior surfaces with no projecting edges or burrs to damage wire insulation and should be sealed against entry of concrete.

(*d*) *Deflections*—Unless otherwise specified on structural drawings and specifications the vertical and lateral deflections of the permanent form between supports should not exceed the following: 1/360 of the span or the maximum tolerances permitted in Section 203. Temporary supports may be used, if necessary, to keep deflection within these tolerances.

506—Forms for prestressed concrete construction

(*a*) *General*—The pretensioning or post-tensioning of the strands or cables may be done with or without the form in place during the prestressing operation.

(*b*) *Design*

1. Where the side forms cannot be conveniently removed from the bottom or soffit form after concrete has set, such forms should be designed for additional axial and/or bending loads which may be superimposed on them during the prestressing operation.

2. Side forms that must remain in place during the transfer of prestressing force should be so designed as to allow for vertical and

horizontal movements of the cast member during the prestressing operation. The form should be designed to minimize the restraint due to the elastic shortening in the prestressing operation. In all cases the restraint to shrinkage of concrete should be kept to a minimum, and the deflections of members due to prestressing force and the elastic deformation of form or falsework should be considered in the design and removal of the forms.

(c) *Construction accessories*—Holddown devices for cables or strands should be provided in the casting bed or forms. All openings, offsets, brackets, and all other items required in the concrete work, should be provided for in the formwork. Bearing plates, anchorage assemblies, prestressing steel, conduits, tube enclosures, and lifting devices shown or specified to be set in concrete must be accurately located with formwork templates and anchored to remain within the tolerances given on contract drawings and specifications. Quality and strength of these accessories should be as specified.

(d) *Tolerances*

1. *Precast prestressed individual members*—Forms for this type of construction should be true to size and dimensions shown on plans and should be constructed and protected from warping so that the finished product will be within the following limits, unless otherwise noted on contract drawings and specifications.

Over-all dimension of members

±1/16 in. per 10 ft

Cross-sectional dimensions

Sections less than 3 in.	±1/16 in.
Sections over 3 in. and less than 18 in.	± 1/8 in.
Sections over 18 in.	± 1/4 in.

Deviations from straight line in long sections

Not more than 1/8 in. per 10 ft length

Deviation from specified camber

±1/8 in. per 10 ft of span

Maximum differential between adjacent units in erected position 3/8 in.

(e) *Special provisions for curing and for safety of workmen*—Where required to allow early re-use of forms, provisions should be made to use such accelerated curing processes as steam curing, vacuum curing, or other approved methods.

Safety shields should be provided at end anchorages of prestressing beds or where necessary for the protection of workmen or equipment against possible breakage of prestressing strands, cables or other assemblies during prestressing or casting operation.

507—Forms for precast concrete construction

(a) *General*—This type of form is used for precast concrete items which may be either load or nonload bearing members for structural or architectural uses.

(*b*) *Construction*—Exterior braces only should be used when exposed metal or filled-in pockets resulting from the use of metal ties would present an objectionable appearance.

To assure uniformity of appearance in the cast members or units, particularly in adjacent units where differences in texture and/or color would be visually apparent, care should be taken that the contact surfaces of forms or form liners are of uniform quality and texture.

Form oil or retardant coatings (nonstaining, if required) should be applied uniformly and in accordance with manufacturers' recommendations for this particular class of work.

2. *Accessories*—It is particularly important in this class of work that positive and rigid devices be used to insure proper location of reinforcement. All openings, cutouts, offsets, inserts, lift rings, and connection devices required to be set in concrete must be accurately located and securely anchored in the formwork.

The finished surfaces of members should be free of lift rings and other erection items where same will be exposed, will interfere with the proper placing of precast members or other materials, or will be subject to corrosion. Such items should be removed in such a manner that no remaining metal will be subject to corrosion.

Quality and strength of these accessories should be as required by contract drawings and specifications, but the lifting devices or other accessories not called for in contract drawings are the responsibility of the contractor.

(*c*) *Tolerances*—Forms must be true to size and dimensions of concrete members shown on the plans and be so constructed that the dimensions of the *finished product will be within the following limits at the time of placement of these units in the structure*, unless otherwise noted on engineer-architect drawings:

Over-all dimensions of members
±1/16 in. per 10 ft, but not to exceed ±1/8 in.

Cross-sectional dimensions

Sections less than 3 in.	±1/16 in.
Sections over 3 in. and less than 18 in.	± 1/8 in.
Sections over 18 in.	± 1/4 in.

Deviations from straight line in long sections
Not more than 1/4 in. per 20 ft

Deviation from specified camber
±1/16 in. per 10 ft of span
Maximum differential between adjacent units in erected position 1/4 in.

(*d*) *Removal of forms*—Precast members or units should be removed from forms only after the concrete has reached a specified strength as

determined by the field-cured test cylinders or beams and job history of concrete curing.

Where required to allow early re-use of forms, provisions should be made to use accelerated curing processes such as steam curing, vacuum curing, or other approved methods.

Methods of lifting precast units from forms should be approved by the engineer-architect.

508—Shoring for composite beam and slab construction

(*a*) *General*—The requirements for this specialized type of shoring should be in the engineer-architect's plans and specifications. The shoring design should comply with the following rules.

(*b*) *Erection*—Where shoring for composite action under total load is called for, it should be placed in position on unyielding supports and wedged up tight against bottoms of structural framing, without causing vertical movement of structural members.

Such shoring must be in place before the casting of concrete is commenced, and must be checked for accuracy of position during the concreting.

(*c*) *Size and Spacing*—Size and spacing of such shoring must be adequate to prevent any deflection of the framing when under the load of the weight of freshly placed concrete, whether the latter is placed continuously or in finite placements.

(*d*) *Camber*—Shims, jacks, or screws may be used at shores to camber the supported beams. In providing camber, allowance must be made for downward deflection of shore supports under load of wet concrete. Where shores rest on mudsills, forms should be watched carefully for settlement and corrective measures must be taken if and as needed [Section 103(b) Item 4].

(*e*) *Removal*—Shores should be removed only after the field-cured cylinder tests and curing operations specified in previous sections indicate to the satisfaction of the engineer that the concrete recently cast has attained the strength required for the composite action, and then only after the stated approval of the engineer. The order of removal of shoring should be approved by the engineer.

This report was submitted to letter ballot of the committee which consists of 20 members; 19 members returned their ballots, of whom 18 have voted affirmatively and 1 negatively.

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From the work of
Comité Européen du Béton

Work of the European Concrete Committee

By FRANCO LEVI

Summarizes recommendations approved by the Comité Européen du Béton (CEB) since its organization in 1953. Among the recommendations reviewed are those on ultimate strength design, T-beams, buckling, cracking, deformation, shearing stress, safety factors, and notation. A brief account of planned activities of CEB is also presented.

■ THE CEB (COMITÉ EUROPÉEN DU BÉTON), came into being in 1953 through the initiative of the "Chambre Syndicale des Constructeurs en Ciment Armé de France," the same association which in 1906 promoted the formulation of the first regulations on reinforced concrete. The first board of directors of CEB, consisting of the president of the Chambre Syndicale, A. Balency-Béarn, and L. Baes, Belgium; H. Rüsch, Germany; E. Torroja, Spain; G. Wästlund, Sweden; and Dr. Nennig, Luxemburg, declared in the bylaws that it was the purpose of the committee to "develop studies of research on scientific and technical questions, such as to improve the design of constructions in reinforced concrete" and "to prepare a number of 'recommendations' with a view toward the formulation of rules applicable to European countries, particular care being given to the introduction of ultimate design." To this end, three representatives from each country in western Europe were invited to become members of the committee. They were selected from among research workers, designers, and contractors particularly interested in the new problems concerning both design and employment of reinforced concrete. In addition to these "active" members, the committee now includes substitute, expert, and corresponding members. In the last few years, the number of countries represented on the committee has been gradually increasing, and now includes the following 19 European countries: Austria, Belgium, Denmark, Finland, France, Western and Eastern Germany, Great Britain, Greece, Netherlands, Italy, Luxemburg, Norway, Poland, Portugal, Spain, Sweden, Switzerland, Turkey,

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and Yugoslavia. Particularly fruitful relationships have been established with scientists in the United States through the creation of a committee for cooperation between the American Concrete Institute and the CEB. The ACI is represented by E. Hognestad, C. P. Siess, and R. C. Reese; while the CEB is represented by A. L. L. Baker, England; A. M. Haas, Netherlands; F. Levi, Italy; H. Rüsch, Germany; E. Torroja, Spain; G. Wästlund, Sweden; A. W. Hill, England; S. Soretz, Austria.

As to CEB, the work is being carried out by a series of commissions of study whose conclusions are subsequently discussed in plenary sessions. So far five plenary sessions have been held: Luxemburg (1953), Fontainebleau (1954), Madrid (1956), Rome (1957), and Vienna (1959). In the course of these sessions, several resolutions were approved, which now constitute a coherent whole covering nearly all of the most important features of the theory of concrete.

The purpose of this paper is to summarize and illustrate the conclusions which have so far been approved by CEB, as reported in the *Bulletin d'Information* No. 19.* The program of activities that CEB has planned for the coming months is also presented.

ULTIMATE DESIGN OF CROSS SECTIONS IN SIMPLE BENDING AND IN COMBINED BENDING AND COMPRESSION

This is the problem to which CEB has given its most careful attention from the beginning. It was logical that ultimate design methods should first be defined in connection with the most important type of loading. Since long discussions have taken place on the definition of the "state of reference," i.e., the state to which the ultimate design had to refer, it seemed logical at first, under the influence of the criteria adopted by the Austrian regulations issued in 1953, to refer the design to a "state of exhaustion," characterized by limit values of deformation in the stressed concrete and the tensile steel, beyond which the beam had to be considered as no longer in use. In practice, "exhaustion" precedes actual rupture under short time loads; the relative load value seemed to correspond with good approximation to that causing rupture under long sustained loading. However, it was soon realized that the use of the definition "state of exhaustion" was impractical.

*Comité Européen du Béton, *Bulletin d'Information* No. 19, Sept. 1959, Edited by the Permanent Secretariat, 3 Rue de Lutece, Paris.

In the first place, the load at exhaustion was difficult to measure experimentally. Since it is difficult to obtain measurements of deformations in the ultimate state, it would always have been necessary to carry out long-time tests, which is obviously inconvenient. On the other hand, the available data, a valuable source of statistical control of methods of calculation, naturally referred to actual rupture tests.

These and other considerations led the committee to take the ultimate state as a reference state. Thus, in the Madrid session in 1956, some recommendations were made concerning the performance of bending and compression tests (see *Bulletin d'Information* No. 19, pp. 21-24). These recommendations were intended to point out to the investigator the essential principles to be followed during the tests so that the results might be interpreted correctly. These recommendations concerned:

The method of loading, including: duration of the test, the setup for applying the loads to the beam, and the determination of the maximum load considered as the ultimate load.

The elements to be included in a report describing a set of experiments: geometrical dimensions of the specimens tested, method of manufacturing, curing conditions, control of the quality of steel and concrete, and measurements recorded during the test.

Also at the session in Madrid, fundamental hypotheses of design were established, and it was decided that the results they led to should be submitted to statistical checking. These hypotheses were:

(1) Plane sections remain plane up to rupture. (The possibility of taking into account a slip of the reinforcement with respect to the concrete was not, however, dismissed.)

(2) A compressive stress block of parabolic shape extending over the whole zone in which the concrete is subjected to shortening, with a maximum stress on the compressed edge of the cross section equal to the strength measured on 15 x 30-cm (6 x 12-in.) cylinders. For vertically cast specimens, the strength is reduced by 10 percent. The contribution to the strength of the tensile zone is completely neglected.

(3) A limiting strain in the concrete on the compressed edge of the cross section equal to 0.0035.

(4) Adoption of simplified stress-strain diagrams to represent the law of steel deformation. For mild steel, the schematic diagram consists of Hooke's straight line with a modulus of elasticity equal to 21,000 kg per sq cm (30,000,000 psi) up to the yield point, together with a line parallel to the strain axis beyond it.(For steel with a high yield point, for which the horizontal part of the diagram is short, the adoption of such a diagram leads to underevaluation of the carrying capacity. Unfortunately it is not possible at the present time to adopt a simplified diagram closer to reality, owing to the practical impossibility of giving a guaranty concerning the length of the horizontal part of the diagram.) For cold-worked steel, Hooke's straight line has been kept up to a load equal to 80 percent of the conventional yield stress, at 0.002 permanent strain. This line is connected by a transition

curve to a straight line starting from the conventional yield stress and having a slope ten times the conventional yield stress itself.

The foregoing hypotheses are referred to by CEB as the Madrid hypotheses, after the session at which they were formulated.

Two trends became evident in the preliminary stages of the session held in Vienna in 1959. On the one hand, extensive statistical studies had been carried out to check the degree of accuracy of calculations founded on the foregoing hypotheses, without, however, dismissing the idea of introducing further possible simplifications. On the other hand, an attempt had been made to begin an accurate study of the factors affecting the behavior of cross sections subjected to flexure to establish a fundamental theory on which practical design methods would be based. These are the two trends that are, in a way, harmonized in the resolution adopted in Vienna concerning simple bending and combined bending and compression (*Bulletin d'Information* No. 19, pp. 12-19). This resolution was based on the following considerations.

Statistical studies designed to define a simplified method

Under the guidance of Moenaert, and with the cooperation of Massonnet, the results of about 1800 tests in simple bending and 400 tests in combined bending and compression were analyzed and the measured ultimate moment or load capacities compared with those computed for use in design on the basis of the Madrid hypotheses, with the following results:

For simple bending of rectangular beams with mild steel tensile reinforcement only: average error of + 8.7 percent (in favor of the experimental result).

For simple bending of rectangular beams with cold-worked steel tensile reinforcement only: average error of + 4.6 percent.

For simple bending of rectangular beams with tensile and compressive reinforcement: average error of + 5.2 percent.

For simple bending of T-beams: average error of + 8 percent.

For combined bending and compression: average error of +7.8 percent.

The root-mean-square deviations in the various instances were between 8 and 10 percent. We then have, following the Madrid hypotheses, a certain coefficient of safety, at least as regards rupture under a load of short duration.

When interpreting these results, we must, however, bear in mind that the errors result only in part from lack of accuracy in the design method. Part of them is to be attributed to the deviation between the strength introduced in the design (as deduced from experiments on test pieces) and the actual strength of the materials in the rupture section. A complementary study carried out in Turin under the author's guidance enabled us to demonstrate that the scattering effect of the hypothesis on which the design was based was less than the scattering effect due

to the different properties of the material. This proves that, in practice, the assumed hypotheses are already sufficiently accurate. It was objected, however, and not without grounds, that the adoption of a parabolic compressive stress block was most unsuitable from a practical point of view, in connection with the design of sections of various patterns, such as T-sections, as well as with the checking of the sections in combined bending and compression. Moenaert and, along the same lines, Torroja and Steinmann, made additional studies with the purpose of determining whether it was possible to substitute for the parabolic diagram a rectangular diagram extending over only a part of the zone subjected to shortening. These studies proved that, in effect, such substitution is possible, both in the field of the so-called perfect bending, characterized by attainment of the yield stress of the tensile reinforcement (assumed to be made of mild steel), and in the field of "imperfect bending," i.e., when the tensile steel is not being fully utilized, or is even compressed. The rules for design resulting from such studies are illustrated in detail in the resolution on p. 1048 and will not be summarized. It seems more interesting, instead, to illustrate the basic theories of the method. The rules applicable to perfect bending (rectangular diagram of height a less than half of the effective depth) are simple and can be obtained from the statistical study which made it possible to establish the dimensions of the rectangle (depth equal to 3/4 of the zone subjected to shortening, and a uniform stress equal to the cylinder strength f_c'. The rupture moment M_u can be expressed in terms of the relative percentage of tensile reinforcement, q, as follows:

$$\frac{M_u}{bd^2 f_c'} = q(1 - 0.5q) \qquad (1)$$

where

b = width of beam
d = effective depth of beam
f_c' = compressive strength of cylinder
q = pf_y/f_c'
p = A_s/bd
f_y = yield strength of tensile reinforcement
A_s = area of tensile reinforcement

Since, on the other hand, we have in this instance: $a/d = q$, the condition $a/d \leq 0.5$, gives $q \leq 0.375$.

For the case of imperfect bending, this method first considers the limiting case of simple compression. In this case, the normal ultimate load P_o on a section of dimensions bt, having both tensile and compressive reinforcement of areas A_s and A_s' is given by the following formula which was also obtained statistically:

$$P_o = 0.75 f_c' bt + f_y(A_s + A_s') \qquad (2)$$

For cross sections with tensile reinforcement only, the relative moment, calculated by taking in account stresses acting on the concrete (with

the exception of the covering of the reinforcement) with respect to the centroid of the reinforcement, is then:

$$\frac{0.75 f_c' bd \frac{d}{2}}{f_c' bd^2} = 0.375$$

We then have exactly the same value of relative moment as for perfect bending, when q reaches the limiting value of 0.5. We then assume that between the limiting case of perfect bending and simple compression, the value 0.375 for the relative moment should remain unchanged. For an intermediate case, when the external stress acts with an eccentricity e with respect to the centroid of the tensile reinforcement (or the centroid of the less compressed reinforcement), the normal ultimate load P_u is then given by an expression of the type:

$$P_u = P_o - KP_u e/t \quad (3)$$

where K, a function of the location and size of the reinforcement, is easily obtained by expressing the condition that the relative moment remains constant. In practice, everything occurs as if the height of the rectangle, equal to the cylinder strength in perfect bending, was gradually decreasing in proportion to the increase of the depth of the compressed zone (and therefore to the decrease of eccentricity), in such a way as to leave the value of the relative moment unchanged. The process can easily be extended to the case of sections with both tensile and compressive reinforcement and of any shape. For sections reinforced with low percentages of cold worked steel, when we wish to take into account the increase in strength which follows the conventional yield point, we should avail ourselves of the hypothesis of plane sections and give due consideration to the shape of the stress-strain diagram for the steel. It is possible, however, to proceed more quickly by calculating the steel stress by the formulas given in the resolution.

Studies designed to establish a basic theory for the bending of reinforced concrete beams

These studies were carried out by Rüsch and his assistants. Since an exhaustive description by Rüsch of his researches has recently been published,* only the principal concepts are outlined here. Rüsch's starting point is to characterize the law of deformation of concrete by thoroughly dependable diagrams, drawn in perfectly defined conditions. For this purpose, he deliberately did not use ordinary machines for testing materials, in which, at best, the rate of application of the load can be regulated only during the first stage of the test, while, in the case of heavy loads, the loading rate varies with each case since the testing process depends on the behavior of the specimen. He, therefore,

*Rüsch, Hubert, "Researches Toward a General Flexural Theory for Structural Concrete," ACI JOURNAL, V. 32, No. 1, July 1960 (*Proceedings* V. 57), pp. 1-28.

built a special machine to permit compression tests during which the rate of deformation is rigorously kept constant.

From tests made in this special machine at strain rates ranging between 0.001 per min and 0.001 in 100 days, Rüsch obtained stress-strain relationships and compressive strengths which varied substantially as a function of the strain rate. The interesting feature of such results, insofar as the behavior of the compressed zone of beams is concerned, lies in the possibility they offer for attributing a different law of deformation to fibers lying at different distances from the neutral axis and thus being strained at different rates for a given rate of load application on the beam. This leads to compressive stress blocks which vary in shape and in magnitude of the maximum stress as a function of the rate of load application on the beam. Rüsch also found that the limiting compressive strain in the outer fiber of the compressed zone varies with the rate of loading, the shape of the cross section, and the location of the neutral axis. These results provide the basis for the statements in the second and third paragraphs of the resolution adopted in Vienna, presented subsequently. The results allow us to study the influence of the rate of loading on the carrying capacity. Such a study reveals that the duration of loading leading to the minimum ultimate strength increases as the neutral axis is lowered in the section. This phenomenon is explained by assuming the existence of two contrasting actions: on the one hand, load applied for a longer time decreases the strength of concrete; on the other, there is a favorable effect caused by redistribution of the strains due to the effect of creep. However, the latter effect cannot take place in sections that are almost entirely compressed, since it is impossible to increase further the area over which the strains are acting. To conclude, in the presence of long-duration loads there occur substantial decreases in the strength (of the order of 25 percent) for sections subjected to simple compression, but only negligible decreases for lightly reinforced sections in bending.

On the whole, Rüsch's theory obviously does not lead to simple design formulas. Nevertheless, he has pointed out that it can be put at the designer's disposal in the form of diagrams as convenient to use as those that can be obtained from various simplified theories. Such diagrams, when based on the most unfavorable loading conditions (i.e., those corresponding to a minimum carrying capacity), do not yield results much different from those obtained when applying the Madrid hypotheses (parabolic diagram, shortening equal to 0.0035, etc.). This result provides a basis for the statements in points four and five of the Vienna resolution. It was previously found that the practical design methods in Section II of the resolution, including further simplification as compared to the Madrid hypotheses, are justified by the statistical studies that were carried out. The test of this resolution follows:

RESOLUTION ON SIMPLE BENDING AND COMBINED BENDING AND COMPRESSION

I. Principles of calculation

1. The Europeen Committee for Concrete considers it necessary to explain as accurately as possible, over a wide range of application, the actual behavior of reinforced concrete structural members and their ultimate strength, in simple bending and in combined bending and compression, for all geometrical shapes of cross section. The conclusive development of this general theory will call for additional research.

2. From a considerable number of test results now available, it appears that the compressive stress distribution diagram is governed in particular by the rate and duration of loading (time effect), the magnitude of the stresses, the quality of the concrete, the position of the neutral axis at failure, and finally the geometrical shape of the section under consideration.

3. It has furthermore been found that there is no generally applicable value for the strain of the concrete at failure (ultimate compressive strain).

4. Despite all these difficulties, it has been demonstrated—thanks to basic theoretical research and statistical interpretations of ultimate load measurements—that, within certain limits of application, the use of simplified stress distribution diagrams (stress blocks) gives sufficiently accurate results.

5. It is recommended that the parabolic stress distribution diagram be employed as a simplified diagram producing sound results, with the assumption that the maximum concrete stress at the extreme compression fiber is equal to the cylinder strength (crushing strength measured on a specimen 30 cm in height and 15 cm in diameter) and that the corresponding strain of the concrete is 0.0035.*

6. Any other simplified stress distribution diagram (stress block)—preferably the rectangular diagram—and also any simplified graph or formula may be used in practice, provided that the results agree within acceptable limits with those obtained with the parabolic diagram or from a large number of tests. Some examples of simplified methods are given in Section II of this resolution.

II. Practical methods of calculation

1. The results of the general theory will be represented in graphs and can shortly be made available to designers for use in the practical analysis of structural members subjected to bending.

2. Furthermore, in accordance with the principles in Section I-6, the collected experimental data show that it is also possible to use, for the analysis of simple bending and combined bending and compression, a rectangular stress distribution diagram for which the uniform concrete stress is taken equal to the cylinder strength f_c' so long as the depth a of this rectangular diagram, which is taken to be 75 percent of the zone subjected to compressive strain, does not exceed half the effective depth d of the section.†

For values of a greater than $0.5d$, up to the case of axial compression, the uniform stress of the rectangular diagram should be so reduced that the resultant moment of the compressive stress in the concrete (in relation to the centroid of the tension reinforcement or the reinforcement least in compression) always remains equal to the moment obtained for $a = 0.5d$. Thus, the effect of the duration and the mode of application of the loads is taken into account.

*It must be recalled that, at previous sessions of the Committee, it was also agreed that "the strain distribution diagram in a cross section could be considered linear up to failure. If need be, slip of the reinforcement in relation to the concrete would have to be taken into account."

†In the case of members which have been concreted in the vertical position, it is suggested that, in compliance with experimental evidence, a reduction of 10 percent be applied to the cylinder strength of the concrete.

In the case of steels with a definite yield point, this enables the main reinforcement to be determined without having to consider the compatibility equation which expresses the condition that plane sections must remain plane.*

In the case of cold-worked steels, if the depth a of the rectangular diagram does not exceed half the effective depth d, the stress f_{su} in the reinforcement at flexural failure generally exceeds the conventional yield point (0.2 percent proof stress) $f_{0.2}$ of the steel. If it is desired to take the resulting increase in the ultimate strength into account, the hypothesis that plane sections remain plane can then be used for determining the strain and the corresponding stress in the steel, bearing in mind that the rectangular compressive stress diagram for the concrete, as defined above, extends over a distance of 75 percent of the depth of the compression zone. In this range, where the depth of the rectangular stress diagram is less than half the effective depth the stress f_{su} can, for simplicity, be taken as being equal to the following value in the case of a cold-worked steel with a 0.2 percent proof stress of about 4000 kg per sq cm (57,000 psi):

$$f_{su} = f_{0.2}\left[1.20 - \frac{1800p}{f_c'}\right]$$

where all the stresses are in kg per sq cm and where the percentage of reinforcement p is referred to the effective depth d. This formula must be used only in the case $f_{su} \geqq f_{0.2}$.†

3. With a view to developing the technical collaboration between the American research organizations and the European Committee for Concrete, the latter adopted the additional resolution that the committee should make a comparison between these simplified methods and the methods proposed by Hognestad and presented to ACI.

T-BEAMS

The resolution adopted in this field is the result of the work of a commission directed by G. Brendel.‡ This commission, created in Rome in 1957, was given the task of studying the possibility of bringing the design of T-beams back within the limits of traditional methods. Its aim was to determine the effective width of the compressive flange so that, by substituting a uniform stress distribution over this effective width for the actual stress distribution, the values of ultimate load and of curvature should remain practically unchanged. In effect, as the calculations for flexure are based on ultimate conditions, the determination

*In general, the compression reinforcement bars may be considered as being subjected to a stress equal to the yield point, provided that they are secured by ties or spirals at fairly close intervals and that they are sufficiently close to the extreme compression fiber, this distance being considered relative to the position of the neutral axis. The errors resulting from this approximation are small.

†For a cold-worked steel whose 0.2 percent proof stress differs from 4000 kg per sq cm, the stress f_{su} may be taken as:

$$f_{su} = f_{0.2}\left[1.28 - \frac{f_{0.2}}{50{,}000} - 0.45\,\frac{f_{0.2}p}{f_c'}\right]$$

This formula is also applicable only in the case where $f_{su} \geqq f_{0.2}$.

‡Since ACI expects to publish material from the work of this commission at a later date, it is only summarized here.

of the effective width should also be based on the same conditions. However, the available experimental data seem to prove that, as the load increases, the width of the compressive flange acting with the beam tends to increase. Therefore, when making calculations in the elastic range, we are on the safe side. The commission consequently decided to carry out their studies for the elastic condition, and thus was able to make use of the experimental and theoretical results that were available in the literature. It was then the chairman's task to provide a synthesis of the available theoretical and experimental data, from which practical rules capable of being conveniently applied could be obtained. When reading the preliminary reports of the resolution adopted in Vienna, we are made aware that such an aim was brilliantly achieved. The work which had then been done, although requiring further research and checking, represented a valuable contribution to the solution of a problem about which the rules in force in the different countries provided inaccurate and often conflicting indications. A number of experimental controls established by Brendel on real beams and plaster models proved, on the other hand, how well the approximations introduced agreed with the observed behavior.

In the resolution, the values of the effective width are given in the form of tables which were formulated by taking into account both the interaction between web and flange through shearing stresses (and considering that the presence of the flange should prevent the free deformation of the web through intervention of shearing stresses along the surfaces of reciprocal contact), and the contribution to strength given by the specific rigidity of the flange itself. The tables are intended for use with symmetrical beams, but it is permissible for the corresponding data to be used also for each side of an unsymmetrical beam. The data refer to simply supported beams made of one or more ribs subjected to distributed loads. However, a correction formula permits one to take into account the local decrease in the cooperating width at the points of application of more or less concentrated loads. For continuous beams, the span between the points of contraflexure is considered. Another correction formula enables one to consider the simultaneous existence of various systems of load. For example, in Fig. 1 and 2 we have reproduced graphically the data relevant to beams characterized by a ratio of span to web thickness of 200. The curves clearly show the great importance of the rigidity of the flange.

The Vienna resolution regarding T-beams, which is summarized below also includes special rules concerning the behavior of the zone adjacent to a simple support, the influence of haunches between the web and flange, and a simplified rule for approximate design, attributing to the effective width on one side of the web the value of 1/10 the span.

RESOLUTION ON T-BEAMS

1. The committee records that, thanks to an exhaustive investigation of the phenomena in the elastic range, its chairman has been able to derive rules for practical application. The committee is accordingly presenting the following recommendations, applicable to symmetrical T-beams, but considers that experimental research should continue, particularly with regard to:

(a) the case where the T-beams are subjected to concentrated loads

(b) the range where the effective width of the T-beams enters into the analysis of the statically indeterminate moments

As regards asymmetrical T-beams, it will be permissible, pending research results, to base the calculations on the recommendations relating to symmetrical T-beams.

2. For freely supported T-beams, having either a single rib or a number of parallel ribs attached to the same slab, the effective width b_e of the compression flange may be determined by using the numerical values given in Tables 1 and 2.

The values indicated are valid for uniformly distributed (or practically uniformly distributed) applied loads. They are also valid for the case of a triangular, parabolic, or sinusoidal distribution of the loading, and also for the case of a constant bending moment.

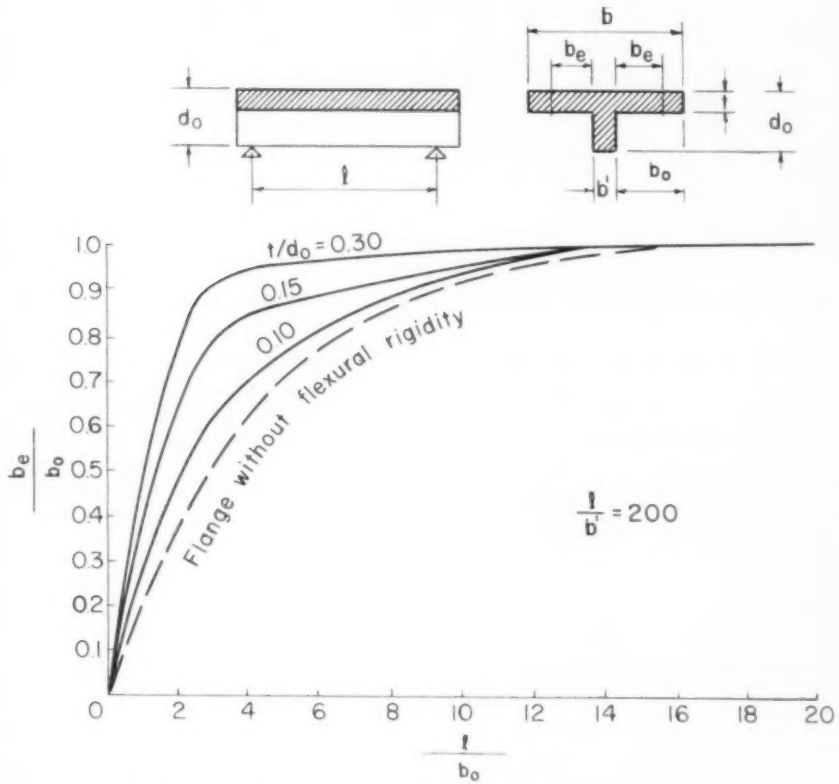

Fig. 1—Effective width of flange at midspan for uniformly loaded, isolated T-beam

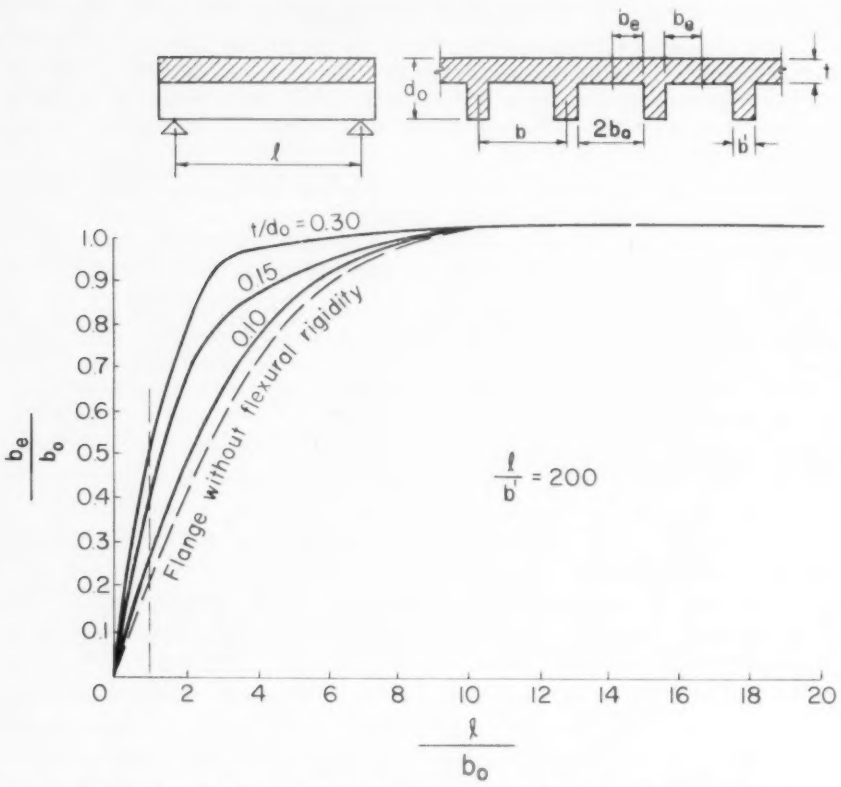

Fig. 2—Effective width of flange at midspan for multiple T-beams all uniformly loaded

On the other hand, if the beam under consideration carries a local load, the value of the effective width b_e of the compression flange should, at the section where the load is located, be reduced in the following proportions with respect to the foregoing values:

	Values of l/b_e	20	10	0
Values of the effective width	for: $a \geq l/10$	$1.0b_e$	$1.0b_e$	$1.0b_e$
	for: $0 < a < l/10$	calculate by linear interpolation		
	for: $a = 0$	$0.9b_e$	$0.7b_e$	$0.5b_e$

l = span of beam between simple supports
b_e = effective width of overhanging portion of compression flange
a = length of zone of application of local load along span of beam

3. For T-beams in which, under a given set of loads, changes of sign occur in the bending moment (e.g., for continuous T-beams) the recommendations contained in Section 2 shall be applicable, provided that the distance between the points of zero bending moment is adopted for the value l of the span.

TABLE I — EFFECTIVE WIDTH OF FLANGE AT MIDSPAN FOR UNIFORMLY LOADED ISOLATED T-BEAM*

$\frac{t}{d_e}$	$\frac{l}{b'}$	Values of b_e/b_e												
		l/b_e												
		0	1	2	3	4	6	8	10	12	14	16	18	20
Flange without flexural stiffness		0	0.18	0.36	0.52	0.64	0.78	0.86	0.92	0.95	0.97	0.98	0.99	1.00
0.10	10	0	0.18	0.36	0.53	0.65	0.78	0.87	0.92	0.95	0.98	0.99	1.00	1.00
	50	0	0.19	0.37	0.54	0.66	0.79	0.87	0.92	0.95	0.98	0.99	1.00	1.00
	100	0	0.21	0.40	0.56	0.67	0.80	0.87	0.92	0.96	0.98	0.99	1.00	1.00
	150	0	0.23	0.43	0.59	0.69	0.81	0.88	0.92	0.96	0.98	0.99	1.00	1.00
	200	0	0.27	0.47	0.62	0.71	0.81	0.88	0.93	0.96	0.98	0.99	1.00	1.00
0.15	10	0	0.19	0.37	0.53	0.66	0.79	0.87	0.92	0.95	0.98	0.99	1.00	1.00
	50	0	0.22	0.42	0.58	0.69	0.81	0.88	0.92	0.96	0.98	0.99	1.00	1.00
	100	0	0.30	0.51	0.66	0.74	0.83	0.89	0.93	0.96	0.98	0.99	1.00	1.00
	150	0	0.36	0.60	0.73	0.80	0.86	0.91	0.94	0.96	0.98	0.99	1.00	1.00
	200	0	0.40	0.65	0.79	0.85	0.89	0.92	0.95	0.97	0.98	0.99	1.00	1.00
0.20	10	0	0.21	0.40	0.57	0.68	0.81	0.87	0.92	0.96	0.98	0.99	1.00	1.00
	50	0	0.30	0.52	0.69	0.78	0.86	0.90	0.94	0.96	0.98	0.99	1.00	1.00
	100	0	0.40	0.65	0.79	0.86	0.89	0.92	0.95	0.97	0.98	0.99	1.00	1.00
	150	0	0.44	0.70	0.85	0.91	0.94	0.95	0.97	0.97	0.98	0.99	1.00	1.00
	200	0	0.45	0.73	0.89	0.93	0.95	0.96	0.97	0.98	0.99	1.00	1.00	1.00
0.30	10	0	0.28	0.48	0.63	0.72	0.81	0.87	0.92	0.96	0.98	0.99	1.00	1.00
	50	0	0.42	0.65	0.83	0.87	0.90	0.92	0.94	0.96	0.98	0.99	1.00	1.00
	100	0	0.45	0.73	0.90	0.92	0.94	0.95	0.96	0.97	0.98	0.99	1.00	1.00
	150	0	0.46	0.75	0.91	0.93	0.95	0.97	0.97	0.98	0.99	0.99	1.00	1.00
	200	0	0.46	0.77	0.92	0.94	0.96	0.97	0.98	0.99	0.99	0.99	1.00	1.00

*For notation see Fig. 1.

4. In the vicinity of a simple support the effective width b_e of the compression flange of a T-beam must not exceed the distance between the support and the section under consideration.

5. In the case where the web is connected to the compression flange by haunches, the actual width b' of the rib should be replaced by an imaginary width b_1, which should be taken to be equal to:

$$b_1 = b' + 2b_e \qquad \text{if} \qquad b_e < d_e$$

$$b_1 = b' + 2d_e \qquad \text{if} \qquad b_e > d_e$$

where: b_e = the width of the haunch
d_e = the depth of the haunch

6. For the case where different sets of loads exist simultaneously, which, if they acted separately, would determine the moments M_1, M_2, ... and the effective widths b_{e1}, b_{e2}, ..., then the effective width b_e of the T-beam should be calculated by the formula:

$$b_e = \frac{M_1 + M_2 + \dots}{\frac{M_1}{b_{e1}} + \frac{M_2}{b_{e2}} + \dots}$$

7. For an approximate calculation of the effective width b_e of a T-beam, it can be assumed as a first approximation that b_e/b_e is proportional to l/b_e, i.e., it can be assumed that

$$\frac{b_e}{b_e} = \frac{1}{10} \cdot \frac{l}{b_e}$$

$$b_e = \frac{l}{10}, \text{ but not more than } b_e$$

TABLE 2 — EFFECTIVE WIDTH OF FLANGE AT MIDSPAN FOR MULTIPLE T-BEAMS ALL UNIFORMLY LOADED*

$\frac{t}{d_o}$	$\frac{l}{b'}$	Values of b_e/b_o							
		l/b_o							
		0	1	2	3	4	6	8	10†
Flange without flexural stiffness		0	0.19	0.38	0.57	0.71	0.88	0.96	0.99
0.10	10	0	0.19	0.38	0.57	0.72	0.88	0.96	0.99
	50	0	0.19	0.39	0.58	0.73	0.89	0.96	1.00
	100	0	0.21	0.42	0.60	0.75	0.89	1.00	1.00
	150	0	0.24	0.45	0.62	0.75	0.90	1.00	1.00
	200	0	0.27	0.48	0.64	0.77	0.90	1.00	1.00
0.15	10	0	0.19	0.39	0.58	0.72	0.89	0.97	1.00
	50	0	0.23	0.44	0.62	0.74	0.90	0.97	1.00
	100	0	0.31	0.53	0.68	0.78	0.91	0.97	1.00
	150	0	0.37	0.61	0.74	0.83	0.92	0.97	1.00
	200	0	0.41	0.66	0.80	0.87	0.93	0.98	1.00
0.20	10	0	0.21	0.42	0.61	0.74	0.90	0.97	1.00
	50	0	0.30	0.54	0.71	0.82	0.92	0.97	1.00
	100	0	0.41	0.66	0.80	0.87	0.94	0.98	1.00
	150	0	0.44	0.71	0.86	0.91	0.96	0.98	1.00
	200	0	0.45	0.74	0.89	0.93	0.97	0.99	1.00
0.30	10	0	0.28	0.50	0.65	0.77	0.91	0.97	1.00
	50	0	0.42	0.69	0.83	0.88	0.93	0.97	1.00
	100	0	0.45	0.74	0.90	0.94	0.96	0.98	1.00
	150	0	0.46	0.76	0.92	0.95	0.97	0.99	1.00
	200	0	0.47	0.77	0.92	0.96	0.98	0.99	1.00

*For notation see Fig. 2.
†$b_e/b_o = 1.00$ for values of l/b_o greater than 10.

BUCKLING

The resolution adopted in this field is the result of the work carried out by the chairman of the corresponding commission of study, A. Aas-Jakobsen. The design method resulting from the study is simple and the field of application extremely wide. In practice, the procedure may be considered as an improvement of the Rankine method, which considers the actual behavior of reinforced concrete structures at ultimate conditions.

To give a clear idea of the criteria followed by Aas-Jakobsen, the bases of the Rankine method are summarized in the following paragraphs.

The maximum stress in an axially-loaded column with unrestrained ends is the sum of two terms: one term, P/bt, corresponding to the normal force; and a flexural term proportional to the bending moment $P\delta$ (δ being the midspan deflection) and inversely proportional to the modulus of resistance bt^2. On the other hand, the deflection is proportional to the square of the length of the column and to the curvature while, finally, the maximum permissible deflection is inversely proportional to the height t of the section. At limit conditions, the second term is then proportional to

$$\frac{P_{cr}}{bt^2} \cdot \frac{l^2}{a} \text{ or } \frac{P_{cr}}{bt} \cdot \frac{l^2}{t^2}$$

where P_{cr} is the critical load. Rupture will therefore be determined by the condition:

$$\sigma_R = \frac{P_{cr}}{bt}\left(1 + K\frac{l^2}{t^2}\right) = \sigma_c\left(1 + K\frac{l^2}{t^2}\right) = \sigma_c(1 + K'\lambda^2) \quad \text{.....(4)}$$

where σ_R is the ultimate strength of the material, K and K' two constants, λ the slenderness ratio, and σ_c the critical stress. As can be seen, Rankine's line of reasoning is in terms of ultimate strength. It is founded, however, on hypotheses based on the laws of perfect elasticity. Aas-Jakobsen follows the same reasoning, but bases his calculations on a consideration of the nonlinear behavior of concrete, that he characterizes by the Madrid equation parabola:

$$\frac{\sigma}{\sigma_o} = \frac{\varepsilon}{\varepsilon_o}\left(2 - \frac{\varepsilon}{\varepsilon_o}\right) \quad \text{.....(5)}$$

where σ_o is the ultimate strength, ε_o the limiting strain and σ and ε the stress and corresponding deformation. Consider then the case of a plain concrete column subjected to an axial load. At the limit, we shall have on the section a stress block of a parabolic shape whose value will vary between the ultimate strength σ_o and a value σ_1. The critical stress can can be obtained by writing

$$\sigma_c = \frac{P_{cr}}{bt} = \sigma_1 + \tfrac{2}{3}(\sigma_o - \sigma_1) = \tfrac{1}{3}(2\sigma_o + \sigma_1) \quad \text{.....(6)}$$

from which we have

$$\frac{\sigma_1}{\sigma_o} = 3\frac{\sigma_c}{\sigma_o} - 2 \quad \text{.....(7)}$$

In the midspan zone of the member, we shall have, at limit conditions, a curvature

$$\frac{1}{r} = \frac{\varepsilon_o - \varepsilon_1}{t} = \frac{\varepsilon_o}{t}\left(1 - \frac{\varepsilon_1}{\varepsilon_o}\right) \quad \text{.....(8)}$$

If we then consider Eq. (5), Eq. (8) can be written:

$$\frac{1}{r} = \frac{\varepsilon_o}{t}\sqrt{1 - \frac{\sigma_1}{\sigma_o}}$$

and, taking Eq. (7) into account:

$$\frac{1}{r} = \frac{\varepsilon_o}{t}\sqrt{3}\sqrt{1 - \frac{\sigma_c}{\sigma_o}} \quad \text{.....(9)}$$

The deflection δ, evaluated by assuming the deformation similar to a sine curve, is then:

$$\delta = \frac{l^2}{\pi^2} \cdot \frac{1}{r} = \frac{l^2}{\pi^2}\frac{\varepsilon_o}{t}\sqrt{3}\sqrt{1 - \frac{\sigma_c}{\sigma_o}} \quad \text{.....(10)}$$

where, in the midspan zone, the resulting moment is:

$$M_c = P_{cr}\,\delta \quad \text{.....(11)}$$

It is still necessary to express the bending-compression ultimate condition in the midspan section. To this end, Aas-Jakobsen chose a formula of the type:

$$P_{cr} = P_o - K P_{cr} \frac{e}{t} = P_o - K \frac{M_o}{t} \quad \text{(12)}$$

identical to Eq. (3) adopted by the European Committee for the simplified method of calculation of cross sections in combined bending and compression.

Eq. (6), (10), (11), and (12), in combination with

$$P_{cr} = \sigma_o b t \quad \text{(13)}$$

establish five equations involving the five quantities σ_c, P_{cr}, M_c, σ_1, and δ, thus solving the problem. In practice, when considering Eq. (12), we notice that the inclusion of buckling can be related to the introduction of an additional moment M_c, for which a simple and easily remembered expression can be given. Let us start from Eq. (10) for the deflection:

$$\delta = \frac{l^2}{\pi^2} \cdot \frac{\varepsilon_o}{t} \sqrt{3} \sqrt{1 - \frac{\sigma_o}{\sigma_o}}$$

and let us consider the Euler value of the critical load:

$$\sigma_E = \frac{\pi^2 E_c}{\lambda^2} = \frac{\pi^2 E_o t^2}{12 l^2}$$

From Eq. (5), we can obtain the value of the modulus at the origin by assuming $\varepsilon = 0$, that is:

$$E_o = 2 \frac{\sigma_o}{\varepsilon_o}$$

Then, by substitution

$$\sigma_E = \frac{\pi^2 t^2 \sigma_o}{6 \varepsilon_o l^2}$$

$$\delta = \frac{t}{2\sqrt{3}} \frac{\sigma_o}{\sigma_E} \sqrt{1 - \frac{\sigma_o}{\sigma_o}}$$

and since, for slender members, the value of the square root can be assumed to be equal to unity, we have in practice

$$\delta \sim 0.3\, t \frac{\sigma_o}{\sigma_E}$$

$$M_o = 0.3\, t \frac{\sigma_o}{\sigma_E} P_{cr} \quad \text{(14)}$$

This is the expression for the additional moment as it appears in the resolution of CEB. Supplementary studies proved in fact that Eq. (14) can be employed with good approximation, not only for the simple

case we have been considering (nonreinforced member with unrestrained ends under axial load), but also for any other condition we may come across in practice, i.e., for:

Nonreinforced members under eccentric normal load
Reinforced members under normal load of small eccentricity
Reinforced members under normal load with large eccentricity
Different conditions of restraint

The procedure can also be extended to two-dimensional structures (plates and shells), whence Paragraph 2.2 of the Vienna resolution. We wish to draw attention to the fact that the adoption by CEB of the method suggested by Aas-Jakobsen was preceded by several different studies. In the first place a comparison was made between the results from the introduction of the additional moment and the data obtained by applying Engesser's method, to nonreinforced concrete columns (when applying Engesser's method, the tangential modulus deduced from the Madrid parabola was naturally considered). The comparison indicated a satisfactory agreement. Subsequently a systematic statistical study was carried out, involving 143 experimental results obtained from the literature, with slenderness ratios varying between 40 and 140 (Baumann, Thomas, Hanson, Rosenstrom, Ramboll, Gehler, Ernst, Gaede). The average ratio between the strength obtained experimentally and the theoretical strength was 1.16, with a coefficient of variation of 15 percent. The commission was also concerned with the effect of creep of concrete, since the available theoretical and experimental data are still scarce in this field. On the basis of the existing theoretical studies (Gaede, Levi) and of some experimental results obtained by Gaede, only a conservative rule was introduced in the resolution, consisting of an increase of the additional moment resulting from the permanent load. The text of the Vienna resolution on buckling follows:

RESOLUTION ON BUCKLING

1. If the characteristics of the material are known, it is possible to calculate correctly the loss of strength of a structure due to its slenderness. This calculation is complex, however. That is why it is necessary in practice to have simplified methods of calculation at one's disposal.

2. The European Committee for Concrete recommends the following simplified method of calculation:

2.1. For the ultimate load design of sections of slender members; e.g., columns, framed structures, or arches; the reduction in strength due to the risk of buckling should be taken into account by introducing an additional moment M_s:

$$M_s = 0.3\, t\, \frac{\sigma_s}{\sigma_B}\, P$$

where:

P = the effective direct force, which is assumed to be calculated in accordance with the theory of first order* and multiplied by the corresponding factor of safety

t = the over-all depth of the section

σ_c = the utilizable strength of the concrete at the extreme compression fiber (this strength corresponds to the reference strength f_c', as employed in the design of short reinforced concrete members subjected to eccentric compression)

σ_E = Euler's characteristic stress

For example, in the case of a strut with a hinge at each end, the value of Euler's characteristic stress is:

$$\sigma_E = E_c \frac{\pi^2}{\lambda^2}$$

where:

E_c = the modulus of elasticity of the concrete (which may be taken as being approximately equal to 1000 f_c')

λ = the slenderness coefficient, the ratio of the length l of the column to the radius of gyration r of the concrete section neglecting the reinforcement

2.2. For the ultimate load design of two-dimensional members; e.g., slabs, diaphragms, or shells; an additional moment M_c should be introduced in each of the principal directions:

$$M_c = 0.3t\sigma_c \left[\frac{P_x}{\sigma x E} + \frac{P_y}{\sigma y E}\right]$$

2.3. If the effect of a sustained load γP has to be taken into account (γ denotes the ratio of the dead or permanent load to the total load), the additional moment should be increased in accordance with the following expression:

$$M_L = (1 + \psi\gamma)M_c$$

The value of the coefficient ψ which takes into account the effect of time should be between zero and one, depending on the magnitude of the anticipated plastic deformations.

3. The Austrian delegation dissented from this proposal for a simplified method of calculation.

VERIFICATION OF THE CONDITIONS OF CRACKING

When a concrete structure is designed for ultimate load, it is obviously necessary to check that its behavior under the service load should satisfy all the requirements pertaining to its normal use. In practice, it is necessary that certain conditions should be observed as regards deformation and cracking. As to cracking, CEB came to formulate, through the work of a commission, under the chairmanship of L. P. Brice, a synthesis of the different methods for the calculation of the spacing and width of cracks in the members submitted to tension or

*That is, not taking into account the effect of the deformations on the stresses in the structure.

bending.* A general theory has been derived which is based on the following hypothesis:

(a) At the location of a crack, all the tension force is taken up by the metal reinforcement.

(b) The maximum spacing between two cracks l_{max} occurs when the tension force, transmitted through bond from steel to concrete, reaches, at half the distance between the cracks, a value slightly less than the tensile strength of concrete.

(c) The crack width is equal to the difference between the variation in length of the concrete and the steel in the region between two adjacent cracks. The maximum width occurs when the distance between the cracks is l_{max}.

On such a basis it had been possible to establish formulas which allowed the spacing and width of the cracks in beams under simple bending or simple tension to be calculated. These formulas contained several coefficients whose values depended on the analytical form attributed to the variation law of the bond in the zone between two neighboring cracks. In the course of subsequent discussions it was realized that the essentially uncertain phenomenon of cracking did not provide a reason for applying in practice complicated formulas containing, in addition, parameters difficult to determine. Hence, simplifications were introduced into the general formula, the most important of which assumes the tensile zone of a beam to be similar to a member under simple tension. This allows the use in all cases of the formulas referring to elements under tension only. The simplified formulas thus obtained show that the crack width depends essentially on the diameter and the shape (smooth or deformed) of the bars, on the steel stress, and finally on the percentage of the tensile reinforcement with respect to the area of the idealized member under axial tension. The practical use of the procedure, nevertheless, still presented two difficulties: some analytical complexity of the formulas, and the random character of the foregoing phenomenon which does not allow the introduction, without due consideration, of the concept of maximum permissible crack. To overcome such drawbacks, the European Committee therefore adopted the following procedure:

(a) Replacement of the theoretical analytical expression with simpler semiempirical formulas.

(b) Transformation of the verification based on comparison between the theoretical width of the cracks and the permissible values to the form of an algebraic inequality.

Such are the concepts on which the first suggestion regarding design is based in the Vienna resolution. The simplified formula adopted to calculate the crack width involves the diameter of the reinforcement, the steel stress, and the percentage of reinforcement referred to a

*To the said synthesis, the studies of Saliger, Wästlund, Johnson and Ostermann, Brice, Watstein and Parsons, Kuuskoski, Clark, and Rüsch can be related.

fictitious member under axial tension, so that the section of concrete surrounding the reinforcement should have its centroid coinciding with that of all the tensile reinforcement. (This definition remains valid for sections in which the reinforcement is distributed almost uniformly and a fictitious member under axial tension can be considered.) The values of the second member of the algebraic inequality were calculated on the basis of limiting widths of the cracks previously adopted by the European Committee. Such values are: 0.1 mm for members of structures subjected to corrosive action; 0.2 mm for nonprotected members of usual structures, and 0.3 mm for protected members of usual structures. Let us point out that the formula contains a safety factor meant to cover the inevitable uncertain element in the design.

The committee also thought it advisable to indicate a second method of design based on a formula suggested by Rüsch, as a result of statistical research. In this case, the area of concrete to which we relate the metal reinforcement to calculate the percentage of steel is the total cross-sectional area of the web in tension. Verification is accomplished again by an algebraic inequality, whose limiting values also consider the three working conditions previously mentioned. Usually, the results which the latter method of checking leads to are more rigorous than those of the former. It will be the task of the Commission on Criteria of Safety to harmonize the two methods. Additional studies are being planned on phenomena that the present resolution is not concerned with, such as the influence of shear, the condition of cracking in the web of deep beams, the duration of application of loads, and thermohygrometric effects. Extensive experimental controls are also anticipated which will allow a better definition of safety conditions. The text of the Vienna resolution on cracking follows.

RESOLUTION ON CRACKING

1. The European Committee for Concrete has carried out a comparative study of the theories for the analysis of tensile and flexural cracking to arrive at a proposed general theory constituting a "synthesis." The results obtained here enabled simplified methods of calculation to be developed, based on the following principles:

1.1. It must be borne in mind that cracking is essentially a random phenomenon whose occurrence depends on a large number of parameters, among which the diameter, the shape, the percentage, and the stress of the reinforcing bars exercise an influence of prime importance.

1.2. The permissible value for the crack width depends on the manner of loading of the structure and on the environment in which it has to serve.*

1.3. The width of the cracks depends principally on their average spacing and on the stress in the reinforcement. In addition, the spacing of the cracks is in itself a function of the shape of the bars and of the ratio between the bar

*It must be borne in mind that the cover over the reinforcement and the quality of the surrounding concrete also exercise considerable influence on the risk of corrosion of the bars. In any case the values proposed for the permissible crack widths assume by implication that the reinforcement is covered by at least 10 mm of good quality concrete.

circumference and the sectional area B of the concrete in which the reinforcement is embedded, considered as a reference section.

By way of example—subject to a statistical comparison of the calculated values for cracking with the experimental results and to investigation of the extent of the influence on the constructional features of structures—the following simplified methods of calculation, applicable to members acting in bending or in tension, can be indicated.*

First proposal for a simplified method of calculation

As the reference section B, we shall consider the section of concrete surrounding the main reinforcement and having the same centroid as the bars collectively.† The corresponding percentage of reinforcement p_1 is equal to the ratio of the cross-sectional area, A_s, of the steel to the reference section B of the concrete, i.e.,

$$P_1 = \frac{A_s}{B} = \frac{\Sigma \pi D^2}{4B}$$

where D is the diameter of the bars.

This simplified method of calculation is valid only for values of p_1 between approximately 2 and 20 percent.

The calculation consists in verifying that, under the working conditions of the structure, the diameter D of the reinforcing bars, the tensile stress f_s in the bars at the crack, and the percentage of reinforcement p_1 satisfy the following condition:

$$(0.40 + 4.50p_1) \frac{D}{P_1} f_s \leq \alpha$$

or, alternatively, if the yield point f_y of the bars and the corresponding factor of safety γ_s are considered:

$$(0.40 + 4.50p_1) \frac{D}{p_1} \frac{f_y}{f_s} \leq \alpha$$

The values of α for smooth and for rough bars and for various working conditions of the structures, in the case where the main tension reinforcement consists of bars of the same diameter, are listed in the appendix to this resolution.

If the bars are not of the same diameter, a sufficiently close approximation will in general be obtained by using the same values of α, p_1 being related to the section of the concrete surrounding the principal tension reinforcement as a whole. The condition thus obtained will apply to the bar having the largest diameter. In this case, however, a more accurate calculation may be carried out, involving a check for each reinforcing bar with its own particular condition of embedment in the concrete. For the purpose of this check the general report of the committee should be referred to.

Second proposal for a simplified method of calculation

For defining the reference section B, the total section of the web is considered, assuming the reference section to be an approximately constant fraction of this total section.

*Certain delegations considered that the simplified methods of calculation for cracking were not applicable to slabs; this question will form the subject of subsequent research by the committee.

†This definition of the reference Section B, which is assumed to be one continuous whole, is valid for sections containing normal layers of reinforcing bars displaying approximately uniform distribution.

(a) *Beams*:—For rectangular sections, the reference section B is taken as being equal to the total section of the web:

$$B = b'h_t$$

where b' is the width of the beam and h_t is the total depth of the beam in tension (i.e., the depth below the neutral axis).

For sections whose geometrical shape does not differ much from a rectangle the value to be adopted for b' is the average width of the section situated in the tension zone.

For I-sections the check for cracking should be applied to the flange, which should be considered as a member in direct tension.

(b) *Members in direct tension*—In the case of members in direct tension, the reference section B is taken to be equal to three times the total section. For example, for rectangular sections we must take:

$$B = 3\,bt$$

The corresponding percentage of reinforcement is:

$$p_2 = \frac{A_s}{B}$$

The calculation consists of verifying that, under the working conditions of the structure, the diameter D of the reinforcing bars and their tensile stress f_s at the crack satisfy the following condition:

$$D \leq \beta \frac{p_2}{f_s^2}$$

or, alternatively, if the yield point of the bars and the corresponding factor of safety γ_s are considered:

$$D \leq \beta \gamma_s^2 \frac{p_2}{f_y^2}$$

The values of β for smooth and for rough bars are given in the appendix to this resolution.

2. The width of the cracks that are liable to occur in the webs of deep beams may exceed that of the cracks occurring in the vicinity in the principal tension reinforcement. Precautions should be taken to reduce these web cracks; it is suggested that longitudinal bars or rectangular mesh reinforcement should be installed along the faces of the web.

The committee will have to study the problem of determining the corresponding reinforcement.

3. The width of the cracks may be influenced by the method or duration of application of the loads (repeated or alternating loads, sustained loads) and also by thermal and humidity effects.

The committee will also have to study the problem as to how these various parameters may be taken into account.

APPENDIX

Values of α (First proposal)

	Members exposed to highly aggressive influences	Ordinary structural members	
		Unprotected	Protected
Smooth bars	2100	4200	6300
Deformed bars	3300	6600	9900

Values of β (Second proposal)

	Members exposed to highly aggressive influences	Ordinary structural members	
		Unprotected	Protected
Smooth bars	*	5×10^6	10×10^6
Deformed bars	2.5×10^6	12×10^6	22×10^6

Units: D in mm, f_y in kg per sq mm.
*The use of smooth bars in members exposed to highly aggressive influences is not recommended.

DEFORMATIONS

The study of this problem had been started under the chairmanship of S. Soretz only a few months before the session to be held in Vienna. The conclusions contained in the resolution are therefore only a preliminary scheme that the commission will have to improve substantially in the future.

For this reason the more important points in the resolution and corresponding appendix will be summarized briefly and the complete text will not be given.

CEB suggests in the first place an approximate method for the calculation of deflections, based on the work of Jaeger. For loads of short duration, the load is divided into two fractions. Up to cracking we calculate the deformation for a homogeneous cross section and with a stiffness equal to:

$$K_I = E_c I_I \qquad (15)$$

$E_c = 18000 \sqrt{R_{28}}$, kg per sq cm, and I_1 is the moment of inertia of the whole section. The corresponding deflection is then:

$$\delta_I = \alpha l^2 \frac{M_I}{K_I} \qquad (16)$$

where M_I is the bending moment at the appearance of the first cracks, and α is a coefficient representing the conditions of restraint and loading. After cracking, the stiffness is taken equal to

$$K'_{II} = 0.75 E_s A_s d^2 (1 - 2q)(1 - \tfrac{2}{3}q) \qquad (17)$$

where $E_s = 2{,}100{,}000$ kg per sq cm is the modulus of elasticity of steel, A_s the cross-sectional area of the reinforcement, d the effective depth, and q the relative percentage of reinforcement. Apart from the reduction coefficient 0.75, such an expression for the rigidity is obtained by assuming the cracked section in the ultimate state. The total deflection will thus be obtained in the form:

$$\delta = \delta_I + \delta'_{II} = \alpha l \frac{M_I}{K_I} + \alpha l^2 \frac{M_{II}}{K'_{II}} \qquad (18)$$

with $M = M_I + M_{II}$. If the deflection thus calculated exceeds the value:

$$\delta_{II} = \alpha l^2 \frac{M}{K_{II}}$$

with

$$K_{II} = E_c A_c d^2 (1 - 2q)(1 - \tfrac{2}{3}q) \quad (19)$$

we assume: $\delta = \delta_{II}$. The resulting moment-deflection relations are shown in Fig. 3.

For long-duration loads, we shall have to consider the effect of shrinkage and creep by multiplying by three the deflection given by the permanent load. A number of corrections are anticipated to permit reduction of the deflection when the permanent load is applied on concrete that has already undergone a long process of hardening.

In Vienna, consideration was also given to maximum permissible values of the deflections. It was not possible, however, to arrive at definite recommendations on this point, inasmuch as the magnitude of the permissible deformations generally depends on the condition of use of the structures; the question does not then pertain to the field of research of CEB. After the Vienna session, CEB contacted the Conseil International du Bâtiment (CIB), an international organization with the purpose of studying the problems concerning the condition of use of structures. It was then decided that the problem of permissible deflections should be studied jointly by CEB and CIB. While awaiting the conclusions of each study, CEB will probably confine itself to establishing limits for the slenderness of the carrying elements.

SHEARING STRESS

The studies of shearing stress have been carried out by a commission under the joint chairmanship of A. Paduart, Brussels, and E. Giangreco, Naples. It was not possible in Vienna to come to practical conclusions in connection with this problem, since the available theoretical and experimental material does not yet account clearly enough for the influence of the numerous factors occurring in the process. The resolution approved at the time contains therefore only some considerations of a general character (e.g., the advice not to utilize beams without web reinforcement) and is followed by a draft for an experimental program of remarkable extent, designed to explain the influence of the following parameters:

Geometrical shape of the section

Type of loading

Percentage of longitudinal reinforcement: its relative conditions of bond and distribution

Type and percentage of web reinforcement

Type of concrete

Conditions of restraint of the beams

After the Vienna session, the work of the commission has been taking a more definite course and consideration has been given to attempting

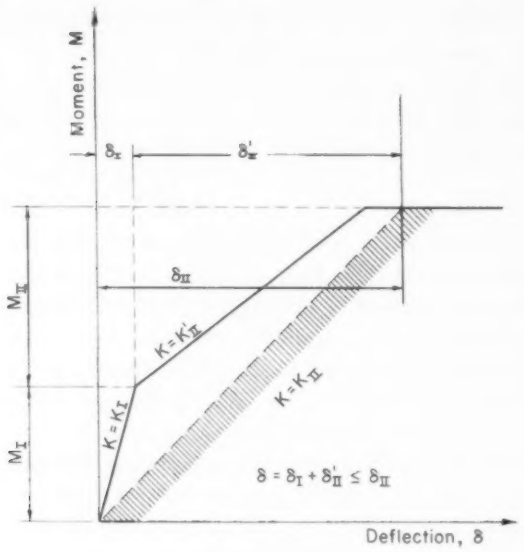

Fig. 3—Moment-deflection relations for short-time loads

a rapid determination of a simplified method of design of temporary character. In practice, it is a matter of defining the problem in relation to a group of simple rules founded on logical considerations that can be formulated in the field of shear failure. The following basic criteria have therefore been adopted.

(a) Sections submitted to high shear and low bending moment

In this field, the ultimate behavior is assumed to be represented with good approximation by the classical truss analogy. An attempt will be made, however, to allow for the contribution that the compressed zone of the concrete can give resistance in shear; it will be assumed also that the slope of the compressed diagonals of the truss analogy can be different from 45 deg. In this field, it is anticipated that two types of failure will be considered, according to the importance of the web reinforcement: destruction by compression of the inclined diagonals of concrete in the web; and rupture of the stirrups.

(b) Sections submitted to high bending moment and low shear

It is assumed that shear here has a disturbing effect, resulting in a reduction of the strength in bending. Such an effect seems to be evaluated, with good correspondence to real physical conditions, by a theory formulated by Paez, Madrid, the results of which are now being compared with the available experimental data. It is also anticipated that in the coming months a series of experiments meant to check the essential points of the simplified theory will be carried out.

REINFORCING STEEL, BOND, ANCHORAGE

This commission under the chairmanship of S. Soretz had limited itself in Vienna to formulating a detailed program of study concerning the problems of anchorage, splicing of reinforcing bars, and necessary

concrete cover. In recent years, such a program has already been started in close cooperation with RILEM (Reunion Internationale des Laboratoires d'Essais des Materiaux). In Vienna the decision previously reached by CEB had also been confirmed, concerning the adoption of simplified stress-strain diagrams for mild and cold worked steel.

SAFETY FACTORS

The resolution in connection with this subject, approved in 1957, in the course of the fourth session held in Rome, was confirmed in Vienna. The fundamental principles, based entirely on statistical considerations of safety, have already been described in the ACI JOURNAL by Torroja, Madrid,* chairman of the corresponding commission of study of CEB. They will therefore be summarized only briefly here.

In practice, it is a matter of dividing the safety factors into several elements and defining the meaning of each element in the clearest possible way. Such a result cannot be achieved unless the methods of the theory of probability are introduced systematically. After the allowable probability of the structure collapsing or becoming unserviceable is established on the basis of precise economic considerations, one must try to determine the dimensions of the structures so that the above mentioned probability is not exceeded, without on the other hand there being an excess on the side of safety. When applied, the process leads to the definition of the "characteristic values" of the strength of the materials and loads. Such characteristic values take not only the average values of such quantities into account but also their coefficients of variation. As such characteristic values consider only a part of the uncertainty, the calculation must be based on further corrected values, called "basic values." The transition from characteristic value to basic value is obtained through the introduction of a safety factor. Its value is caused to vary depending on the particular conditions of the problem under consideration (type of material, accuracy in execution, importance of damage in the case of collapse, etc.). Later the structure will be proportioned so that the elements subjected to basic loads do not exceed a limiting state in which the material reaches "basic resistance."

It may be of interest to remark that the suggestions of CEB proved similar to the fundamental principles followed by the compilers of the regulations on reinforced concrete of USSR; which have been in force since 1955. Let us add that the chairman of the commission, Professor Torroja, has recently formulated further proposals concerning the problem of safety with respect to cracking and elastic buckling.

* "Load Factors"—International Council for Building Research, E. Torroja, chairman, ACI JOURNAL, V. 30, No. 5, Nov. 1958 (*Proceedings* V. 55), pp. 567-672.

NOTATION

The Commission formulated the following general principles:*

The use of Latin capital letters for representation of surfaces and relative statical quantities (statical moments, moments of inertia, etc.) as well as of forces, moments, support reactions.

The use of Latin small letters for lengths and distributed loads.

The use of Greek small letters for tensions and nondimensional coefficients.

The use of Greek capitals is not advised.

It is recommended that the following suffixes be used:

a for steel
b for concrete
e for the yield point
0.2 for the conventional limit of elasticity
r for rupture

Tensions and compressions will, generally speaking, be distinguished by their algebraic sign. If need be, a dash should be added to any symbol to indicate compression.

A minimum use of multiple suffixes is particularly recommended.

On this basis a series of about two hundred notations was prepared. These notations will systematically be used in the activities of the European Committee and will subsequently be brought to the attention of the appropriate committee of the International Standards Organization.

OTHER ACTIVITIES

To conclude, consider briefly the other decisions reached in Vienna.

The most important of them concerns the creation of a special commission under the chairmanship of N. Esquillan, Paris, charged with preparing a list of tentative recommendations constituting the basis of an actual unified design code. These recommendations shall contain in the first place all of the resolutions already adopted by CEB. As to the subjects still under discussion, the most up-to-date regulations to be found in the national regulations in force in the various countries shall be adopted.

In Vienna, two new work groups were set up: one on plates, the other on limit design of hyperstatic structures. A. M. Haas, Delft (with the cooperation of G. Steinmann, Geneva) was appointed chairman of the former; and A. L. L. Baker, London, of the latter. Both these commissions have already carried out useful preparatory work.

In the organizing field the following steps have also been taken concerning the establishment of close cooperative relationships with sev-

*Editor's Note: The CEB system of notation has not been followed in this paper to make it easier for American readers.

eral international associations whose purposes are similar to those of the European Concrete Committee:

(a) To take part in the activities of the Comité de Liaison entre Associations Internationales de Génie Civil. This committee, which was created at the end of 1958 on the initiative of CEB, groups together the representatives of CEB and the following organizations:

International Association for Bridge and Structural Engineering (IABSE)
Reunion Internationale des Laboratoires d'Essai des Materiaux (RILEM)
Federation Internationale de la Precontrainte (FIP)
International Association for Shell Structures (IASS)

The purpose of this liaison committee is to coordinate the research studies carried out by the different associations, to make the dates of the scientific meetings suitably agree, to establish close cooperation in organizing colloquia and symposia and finally to foster the creation of joint committees for the investigation of problems of common interest.

(b) To create a collaboration committee between ACI and CEB. A decision on this point was reached after a friendly exchange of opinions between the board of directors of CEB and the representatives of the United States attending the session held in Vienna as observers, E. Hognestad, C. P. Siess, and R. C. Reese. In recent months the committee has already done extremely useful work, both through the exchange of scientific information and through participation of ACI representatives in the activities of the different commissions of study. Both groups intend to develop such relationships and overcome the difficulties of distance in the best possible way.

(c) To invite the USSR Academy of Architecture to have its own observers take part in the activities of CEB. Following this invitation a Russian observer attended the meetings of the commissions of study held recently in Rome.

(d) To create a group of Corresponding Members both collective and individual so that they may receive the bulletins and send the commissions their remarks on the work that is being carried on.

(e) To extend the commissions of study through appointment of "experts" by the supervisory organs of CEB.

COMMISSION MEMBERSHIP

After reorganization the commissions of study have the following membership:

Commission I — Tentative Recommendations

N. Esquillan, France, chairman
A. Balency-Béarn, France
E. Bornemann, Germany
F. Correia de Araujo, Portugal
H. Granholm, Sweden
P. Moenaert, Belgium
A. Paez, Spain
R. C. Reese, United States
Y. Saillard, France
G. Steinmann, Switzerland
F. G. Thomas, England

Commission II—Reinforcing Steel, Bond and Anchorage

S. Soretz, Austria, chairman
J. Ferry-Borges, Portugal
E. Giangreco, Italy
K. Hajnal-Kónyi, England
P. Lebelle, France
H. Louis, Belgium
A. Paez, Spain
G. Rehm, Germany
Y. Saillard, France

Commission III — Bending and Compression

IIIa Scientific Research

H. Rüsch, Germany, chairman
A. Aas-Jakobsen, Norway
R. Chambaud, France
H. Granholm, Sweden
E. Hognestad, United States
P. Moenaert, Belgium

IIIb Simplified Methods

P. Moenaert, Belgium, chairman
A. L. L. Baker, England
R. Chambaud, France
R. C. Reese, United States
H. Rüsch, Germany
G. Steinmann, Switzerland

Commission IV — Cracking and Deflections

IVa Cracking

L. P. Brice, France, chairman
A. Efsen, Denmark
J. Ferry-Borges, Portugal
A. M. Haas, Netherlands
Y. Saillard, France
S. Soretz, Austria
G. Wästlund, Sweden

IVb Deflections

S. Soretz, Austria, chairman
L. P. Brice, France
A. Efsen, Denmark
A. M. Haas, Netherlands
A. Hillerborg, Sweden
J. R. Robinson, France
J. C. Maldague, France

Commission V — Shear

A. Paduart, Belgium, chairman, main committee and, subcommittee on general theory
E. Giangreco, Italy, chairman, subcommittee on tests and simplified design method
R. Chambaud, France
A. Paez, Spain
J. R. Robinson, France
H. Rüsch, Germany
C. P. Siess, United States
S. Soretz, Austria
G. Steinmann, Switzerland
F. G. Thomas, England
J. C. Maldague, France

Commission VI — Safety

E. Torroja, Spain, chairman
N. Esquillan, France
H. Rüsch, Germany
F. G. Thomas, England

Commission VII — Notations

S. Soretz, Austria, chairman
F. Levi, Italy
Y. Saillard, France
F. G. Thomas, England

Commission VIII — Buckling

A. Aas-Jakobsen, Norway, chairman
E. Giangreco, Italy
K. Jäger, Austria
K. Kordina, Germany
J. R. Robinson, France
G. Wästlund, Sweden

Commission IX — T-Beams

G. Brendel, Germany, chairman
A. M. Haas, Netherlands
M. Hartenbach, Switzerland
P. Lebelle, France
H. Rüsch, Germany

Commission X — Slabs

A. M. Haas, Netherlands, chairman

Xa Ultimate Design

G. Steinmann, Switzerland, chairman
G. Franz, Germany
K. W. Johansen, Denmark

C. J. Louw, Netherlands
Ch. Massonnet, Belgium
Y. Saillard, France
R. H. Wood, England
C. P. Siess, United States

Xb Flat Slabs

A. M. Haas, Netherlands, chairman
G. Brendel, Germany
K. Jäger, Austria
P. Lebelle, France
H. Nylander, Sweden
C. P. Siess, United States

Commission XI — Statical Indeterminancy

A. L. L. Baker, England, chairman
C. Benito, Spain
F. Correia de Araujo, Portugal
Y. Guyon, France
K. Jäger, Austria
F. Leonhardt, Germany
C. J. Louw, Netherlands
G. Macchi, Italy
A. H. Mattock, United States

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Title No. 57-50

Investigation of Bond in Beam and Pull-Out Specimens with High-Yield-Strength Deformed Bars

By ROBERT G. MATHEY and DAVID WATSTEIN

Bond strengths were determined in 18 beam and 18 pull-out specimens with deformed reinforcing bars having a nominal yield strength of 100,000 psi. The lengths of embedment ranged from 7 to 17 in. for #4 bars and from 7 to 34 in. for #8 bars. The bond strength was found to decrease with increases in the length of embedment for a bar of a given size. The bond also decreased with an increase in the bar diameter for a given length-diameter ratio. Bond failures were obtained in all beams containing #8 bars; with #4 bars, both tensile and bond failures were observed.

The ultimate bond stresses in the pull-out specimens agreed in general with the values obtained in beams with #4 bars. However, for #8 bars the bond strengths in pull-outs were significantly greater than the values obtained with beams.

It was found advisable to use either a loaded-end slip of 0.01 in. or a free-end slip of 0.002 in. to define "critical" bond stresses, depending on which of these slips developed first. Bond stresses corresponding to these values of slip were sufficiently low to insure that under-reinforced beams designed on the basis of these criteria would fail by yielding of reinforcement.

■ THE INCREASE IN THE AVAILABILITY of commercial grades of high-yield-strength deformed bars has stimulated a keen interest among designers in the possibility of using higher working stresses in tensile reinforcement. It is well known that the resisting moments of under-reinforced flexural members are dependent primarily on the total yield strength of the reinforcement,[1,2] and if failure by yielding in tension were the only method of failure possible, immediate increases in design stress in the reinforcement would be justified. However, the possibility of failure by diagonal tension and bond is enhanced by higher stresses in the tensile reinforcement. Therefore, an investigation was undertaken at the National Bureau of Standards to determine the effect of magnitude of these stresses on the bond of deformed bars. This study was supported by the Committee on Reinforced Concrete Research of the American Iron and Steel Institute.

The test procedures used in this study were essentially those recommended by ACI Committee 208, Bond Stress.[3] Bars of two sizes were

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David Watstein, chief of the Structural Engineering Section, National Bureau of Standards, has written numerous papers on concrete research and his 1953 ACI Journal paper was awarded the Wason Medal for Most Meritorious Paper. An ACI member since 1941, he is chairman of ACI Committee 208, Bond Stress, and is a member of Committees 209, 339, and 716 on the subjects of volume changes, allowable stresses in reinforcement, and high-pressure steam curing.

used at several lengths of embedment. The nominal yield strength of the reinforcement was 100,000 psi. Preliminary tests of pull-out specimens were included in this program to determine the approximate maximum length of embedment which could be used in beam specimens and to compare the bond values developed in the two types of test specimens.

MATERIAL AND SPECIMENS

Reinforcement

The tensile reinforcement in the pull-out and beam tests consisted of #4 and #8 deformed bars. A 3 ft length of each bar size was tested to rupture in tension. The yield strengths determined by the "0.2 percent offset" method were 114,700 and 97,000 psi, respectively, for #4 and #8 bars. While these bars did not exhibit a well-defined yield point, their stress-strain relationships were linear up to a stress of about 100,000 psi for the #4 bars and approximately 80,000 psi for the #8 bars. Tensile properties of the bars are given in Table 1, and typical stress-strain curves are shown in Fig. 1. The properties of deformations were determined from one coupon of each bar size and are given in Table 2. Deformations of the reinforcing bars met the requirements of ASTM A 305-56T. Typical reinforcing bars used as tensile reinforcement in the pull-out and beam specimens are shown in Fig. 2.

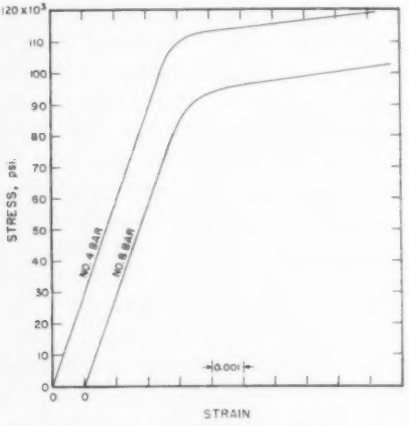

Fig. 1—Typical stress-strain characteristics of reinforcing bars

Concrete

The concrete was made from a mix of cement, sand, and gravel proportioned approximately 1:3.5:4.4, by weight. Beam specimens were made with Type I cement and the pull-out specimens were made with Type III cement. The sand and gravel were siliceous aggregates from White Marsh, Md. The maximum size of the coarse aggregate was 1 in. and the fineness modulus of the sand was 2.71. Water content of the concrete was about 8.2 gal. per sack of cement and the slump ranged from 3 to 6 in. All concrete was mixed in a tilting drumtype mixer

TABLE 1 — PROPERTIES OF REINFORCING BARS

Bar size	Area, A_s, sq in.	Perimeter, Σ_o, in.	Yield* strength, f_y, psi	Proportional limit, psi	Tensile strength, psi	Modulus of elasticity, E_s, 10^6 psi	Elongation in 8 in., percent
#4	0.194	1.561	114,700	102,800	137,000	29.6	5.0
#8	0.787	3.142	97,000	78,000	124,000	28.4	6.8

*Yield strength was determined by the "0.2 percent offset" method.

of 6 cu ft capacity and placed with the aid of a laboratory type internal vibrator.

For each beam specimen and for each pair of pull-out specimens, three standard 6 x 12-in. cylinders were cast. The cylinders were stored and cured in the same manner as the specimen, and were tested the same day as the specimen to determine the compressive strength of the concrete. The average compressive strength obtained from tests of standard 6 x 12-in. cylinders was 3910 psi at 28 to 30 days for the beam specimens and 4150 psi at 14 to 15 days for the pull-out specimens. The average values for three cylinders tested for each specimen are given in Tables 3 and 4.

Beam specimens

The specimens listed in Table 3 and shown in Fig. 3 were reinforced concrete beams having a cross section of 8 x 18-in. and an effective depth of 16 in. The beams were essentially of the type described in the report of the ACI Committee 208.[3] The method of support was modified by widening the ends of the beams into a T-shape to permit moving the reactions some distance away from the test bar so as not to confine the concrete at the supports. The recess exposing the bar was reduced to 3 in. in length from 6 in. specified in ACI 208-58, and the span was increased to 88 in. to permit the use of longer embedment lengths required to develop the high yield strength of reinforcement. It was possible to reduce the length of the recess because of the modification in instrumentation for measuring steel strains adopted in this study. It is believed that the shorter length of recess was beneficial in that it did not disrupt the continuity of the beam at the load point to the same extent as the 6-in. notch.

The web reinforcement consisted of #4 bars made into vertical stirrups and spaced 4 in. apart throughout the shear span. The arrangement of the reinforce-

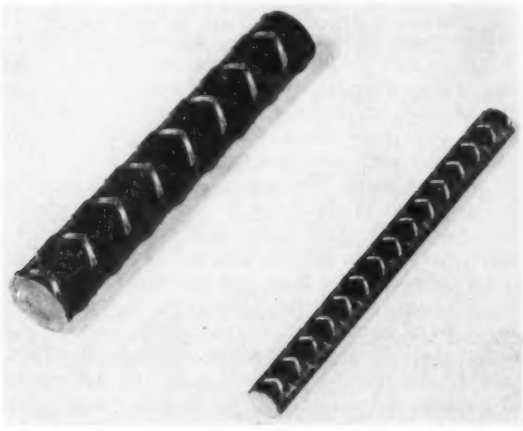

Fig. 2—Views of #8 and #4 reinforcing bars

TABLE 2 — PROPERTIES OF DEFORMATIONS

Bar size	Width of gap, in.	Average spacing, in.	Average height, in.	Average projected length, in.	Bearing area, sq in. per in.
#4	0.073	0.221	0.031	1.26	0.176
#8	0.210	0.408	0.056	2.36	0.323

ment, dimensions of the beam, and method of loading are illustrated in Fig. 3.

The lengths of embedment of the #4 bars ranged from 7 to 17 in. while those of the #8 bars ranged from 7 to 34 in. The actual length of embedment of the tensile reinforcement was also the distance from the support to the nearer concentrated load. The portion of the bar between the support and the end of the beam was covered with rubber tubing to eliminate bond in the overhanging portion of the specimen. Beams with lengths of embedment ranging from 21 to 34 in. were reinforced in compression and contained stirrups throughout the length of span.

Pull-out specimens

The pull-out specimens described in Table 4 were 10 x 10-in. in cross section with the reinforcing bar concentric with the longitudinal axis of the specimen, and the lengths of embedment corresponded to those of the beam specimens. Splitting of the concrete was forestalled by reinforcing the specimens with a cylindrical cage of 2 x 2—12/12 welded wire fabric. The cages had a diameter of 8 in., extended the length of the specimen, and were concentric with the reinforcing bar. The instrumentation and the manner of loading are illustrated in Fig. 4.

Fabrication and curing of the specimens

The reinforcement for the beam specimens was fabricated in the laboratory and assembled into units before it was placed in the forms. Specimens were cast in steel forms with the tensile reinforcement near the bottom. After 5 days they were removed from the forms and moist-cured until 2 days prior to testing at 28 to 30 days.

The pull-out specimens were cast in wooden forms with the reinforcing bar in a horizontal position. After 1 day they were removed from the forms and moistcured until 1 day prior to testing at 14 to 15 days.

TESTING PROCEDURE

The beam specimens were tested as simply supported beams over a span of 7 ft 4 in. in a 600,000 lb capacity hydraulic testing machine. The manner of loading and location of strain gages are illustrated in Fig. 3. Steel rockers supported the beams and the load was applied through a steel loading beam with attached knife edge assemblies. The forces were distributed at the load and reaction points through steel bearing plates 3½ in. wide that covered the entire width of the specimen. The bearing plates at the reaction points were 1 in. thick and were located 7¼ in. from the test bar. A bearing plate, 1½ in. thick, was under one knife edge assembly and under the other, three 1/2 in. diameter steel rollers rested on a 1-in. bearing plate. The load points were directly over the outer edges of the 3-in. notches which exposed the bar at the loaded ends of the embedded lengths. Strains in the tensile reinforcement 7/8 in. from the inner edge of the 3-in. notches were measured with bonded wire electrical resistance strain gages attached at diametrically opposite points at the horizontal midplane of the longitudinal bar. The deformations were filed off for a length of about 3/4 in. to facilitate the placement of these gages. The gages were of

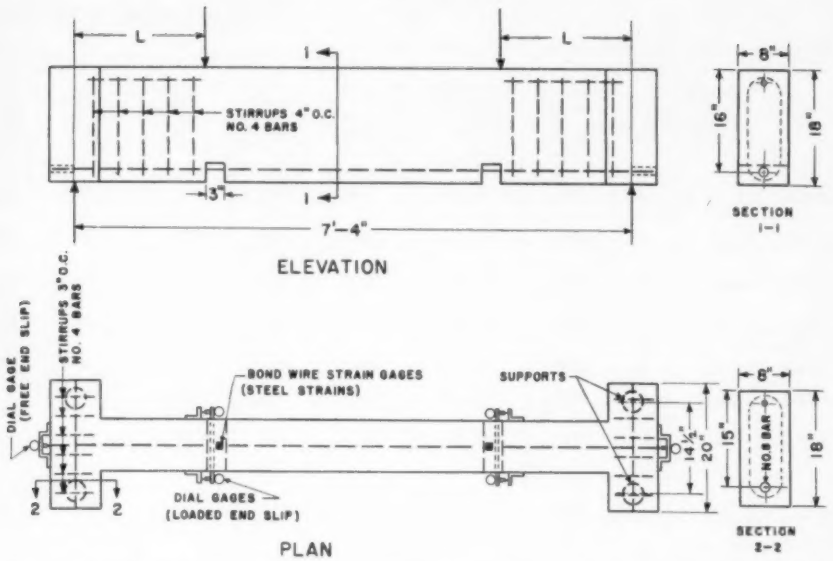

Fig. 3—Details of beam specimens, testing arrangement, and location of strain gages

Type A-7, with a gage length of 1/4 in., and were attached with epoxy resin cement. It is noted that the use of bonded wire strain gages constituted a departure from ACI 208-58 which calls for the use of micrometer dial gages for measurement of strain in the reinforcing bars.

A yoke was clamped into drilled contact points in the exposed bar located 1/2 in. from the outer edges of the notches. A pair of dial gages attached to each yoke permitted measurement of slip of the bar directly under the concentrated loads. The gages bore on steel angle clips attached with screws to the concrete. In determining the slip at the loaded end, the gage readings were corrected for the extension of the reinforcing bar in the distance between the attachment of the yoke and the outer face of concrete of the 3-in. notch. Slip at the free end was measured with dial gages held in position by steel yokes that were attached to the concrete by screws. The micrometer dial gages used in measurement of slip were graduated to 0.001 in. and had a range of 1 in.

The load was applied in increments of 5000 lb at a rate of 4000 lb per min. The following measurements were recorded after each increment of applied load: the strains in the reinforcement, slip at both loaded ends of the bar, and slip at both free ends of the bar. In addition to these measurements, the location and extent of cracks were recorded immediately after the application of each increment of load.

Pull-out specimens were tested in a 200,000 lb capacity universal electromechanical testing machine. The specimen, loaded as shown in Fig. 4, was seated on a leather cushion on two segments of a 2-in. base plate attached to a spherical bearing block. Slip of the bar was measured with 0.001 in. micrometer dial gages and estimated to 0.0005 in. At the loaded end of the specimen, two dial gages were attached to a steel bar that was fastened to the face of the

TABLE 3 — DATA OBTAINED

Beam No.	Length of embedment, in.	Bar size No.	Compressive strength of concrete, f_c', psi	Maximum computed steel stress, psi	Maximum computed bond stress, u_m, psi	Adj.* u_m, psi	Maximum slip observed at free end, in.	Bond stress corresponding	
								Loaded end slip of 0.01 in., u_1, psi	Free end slip of 0.002 in., u_2, psi
4-7-1	7	4	4265	92,100	1636	1503	†	1285	1327
4-7-2	7	4	4210	88,600	1572	1464	0.66	1259	1235
4-10.5-1	10.5	4	3880	115,200	1364	1378	0.008	841	1215
4-10.5-2	10.5	4	4055	115,000	1361	1316	0.37	862	1123
4-10.5-3	10.5	4	3675	113,300	1341	1431	0.43	869	1070
4-14-1	14	4	3865	121,300	1077	1092	0.002	609	—
4-14-2	14	4	3710	100,400	892	943	0.41	573	825
4-17-1	17	4	3880	120,700	882	891	0.35	535	677
4-17-2	17	4	3710	127,400	931	984	0.016	497	—
Avg			3920						
8-7-1	7	8	4005	28,600	1023	996	0.59	955	803
8-14-1	14	8	3585	33,400	598	651	0.49	—	561
8-14-2	14	8	4055	42,500	760	731	0.08	710	722
8-21-1	21	8	4235	61,800	737	679	0.19	702	537
8-21-2	21	8	3495	53,200	635	709	0.18	—	467
8-28-1	28	8	4485	77,200	691	601	†	588	637
8-28-2	28	8	3700	71,800	643	678	0.28	411	512
8-34-1	34	8	3745	92,100	678	706	0.75	505	574
8-34-2	34	8	3765	89,700	661	685	0.34	454	552
Avg			3900						

* u_m and u_o were adjusted by multiplying by a factor $\overline{f_c'}/f_c'$, where $\overline{f_c'}$ is the average concrete strength of a group of beams containing the same size bar.

† A large slip developed, but was not recorded.

WITH BEAM SPECIMENS

to: Free end slip of 0.005 in., u_s, psi	Critical bond stress, u_c,‡ psi	Adj. u_c,* psi	Steel stress corresponding to u_c $(f_s)_c$, psi	u_c § $(f_s)_c$	$\frac{u_m}{2}$, psi	Adj.* $\frac{u_s}{2}$, psi	Mode of failure
1520	1285	1181	72,350	0.01777	818	540	Bond; excessive slip at both ends of bar; no crack within shear span.
1437	1235	1150	69,600	0.01777	786	575	Same as above; spalling of concrete at loaded end
1250	841	850	71,000	0.01185	682	425	Yielding of reinforcement; no crack within shear span
1265	862	833	72,800	0.01185	681	417	Bond; excessive slip at both ends of bar; no crack within shear span
1216	869	926	73,400	0.01185	671	463	Same as above
—	609	618	68,600	0.00888	539	309	Yielding of reinforcement; no crack within shear span
—	573	605	64,550	0.00888	446	303	Bond; excessive slip at both ends of bar; no crack within shear span; spalling accompanied by horizontal crack
749	535	540	73,200	0.00731	441	270	Bond; a wide crack developed within shear span, causing an anchorage failure in the region between the crack and the free end; excessive slip at free end
—	497	525	68,000	0.00731	466	263	Yielding of reinforcement; one narrow crack within shear span
964	803	782	22,500	0.03569	512	391	Bond; cracks formed within shear spans; loaded-end slip at one end was low due to a wide crack
595	561	610	31,350	0.01790	299	305	Bond; wide cracks formed within both shear spans; causing low values of loaded-end slips
—	710	683	39,700	0.01790	380	342	Bond; cracks of moderate width formed within shear spans at both ends
618	537	495	45,050	0.01192	368	248	Bond; crack of moderate width in one shear span and a wide crack in the other, causing a low value of loaded-end slip at that end
527	467	521	39,150	0.01192	318	260	Bond; wide cracks formed at both ends causing low values of loaded-end slip and large free-end slips
—	588	511	65,700	0.00895	346	256	Bond; wide cracks developed in both shear spans accompanied by moderately large loaded-end slips
578	411	433	45,950	0.00895	322	217	Bond; cracks of moderate width in both shear spans; loaded-end slip was low and free-end slip was high at one end
618	505	526	68,550	0.00737	339	263	Bond; wide crack accompanied by spalling at one end; cracks of moderate width at other end
612	454	470	61,600	0.00737	331	235	Bond; wide crack at one end and moderately wide crack at other end; with large free-end slips at both ends

‡ Critical bond stress u_c is the lesser of the two values of bond stress corresponding to a loaded-end slip of 0.01 in. and a free-end slip of 0.002 in.

§ Steel stress f_s corresponds to u_c.

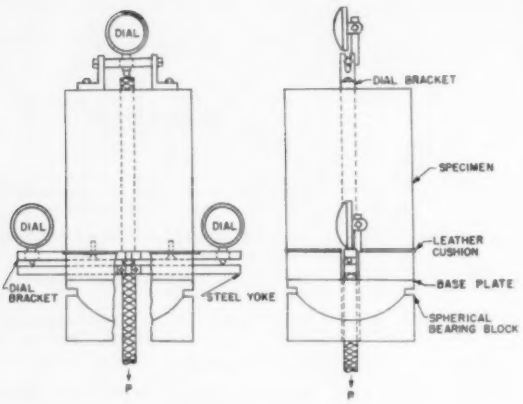

Fig. 4—Details of pull-out specimens, testing arrangement, and location of dial gages

concrete by bolts secured into inserts cast in the specimen. The gages bore on a steel yoke fastened to the reinforcing bar approximately 7/8 in. below the surface of the concrete. The bar supporting the dial gages and the yoke were free to move in the recess in the base plate. The average of the two gage readings indicated the amount of movement of the point on the reinforcing bar at which the yoke was attached with reference to the face of the concrete. In determining the slip at the loaded end, the gage readings were corrected for the extension of the reinforcing bar in the distance between the point of attachment of the yoke and the surface of the concrete. The slip at the free end was measured directly from a gage which bore on the exposed end of the bar, and was mounted on a support that was attached to the top face of the concrete by bolts secured into inserts cast in the specimen.

RESULTS AND DISCUSSION

Bond failure in the beam tests is defined as a failure accompanied by excessive slip at the free end of the bars. Excessive slip is associated with a rapid increase in the movement of the free end of the bar with only a slight increase in the applied load.

Bond failures developed in all beams containing #8 bars and in six of the nine beams containing #4 bars. As can be seen in Fig. 5 showing typical views of beams after test, failures were accompanied by longitudinal splitting along the tensile face of the concrete and by formation of slightly inclined cracks within the shear span in most cases. All beams with #8 bars developed such cracks, while only two beams with #4 bars embedded 17 in. exhibited inclined cracks. It was also noted that for the shortest length of embedment of #8 bars (7 in.), the cracks within the shear span were nearly vertical; however, these cracks became more inclined with an increase in the length of the shear span.

Table 3 presents the values of free-end slip observed after the maximum load on the beam specimen was passed. In some cases, as in Beams 4-10.5-2, 4-10.5-3, and 4-17-1, the bar was stressed to its yield strength

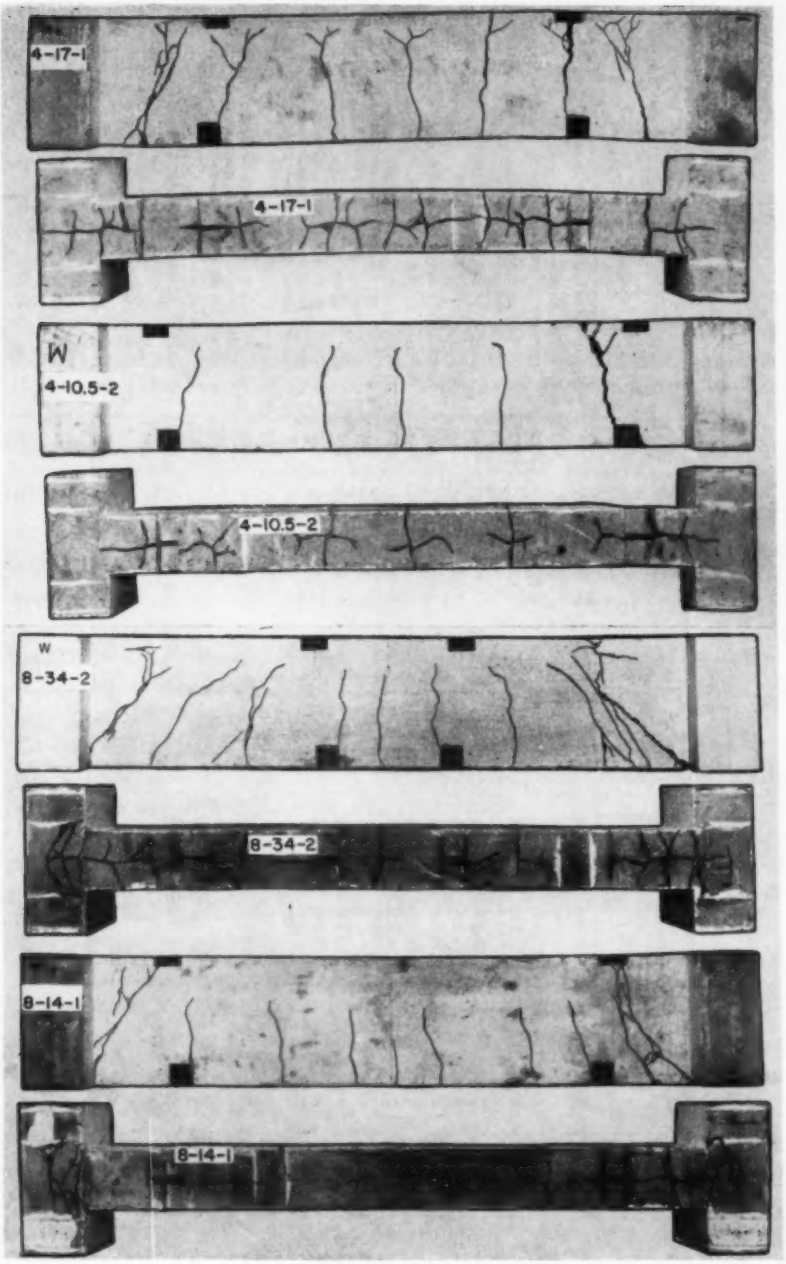

Fig. 5—Views of typical beams after test

TABLE 4 — DATA OBTAINED WITH PULL-OUT SPECIMENS

Pull-out No.	Length of embedment, in.	Bar size No.	Compressive strength of concrete, f_c', psi	Maximum computed steel stress, f_s, psi	Maximum computed bond stress, u_m, psi	Maximum slip observed at free end, in.	Bond stress corresponding to: Loaded-end slip of 0.01 in., u_1, psi	Free-end slip of 0.002 in., u_2, psi	Free-end slip of 0.005 in., u_3, psi	$\frac{u_m}{2}$, psi	Mode of failure
P4-7-1	7	4	4370	86,900	1542	0.21	760	680	890	771	Bond; small transverse crack at loaded end
P4-7-2	7	4	4370	91,200	1620	0.16	790	580	880	810	Same as above
P4-10.5-1	10.5	4	4145	123,700	1464	0.003	790	1310	—	732	Yielding of reinforcement; no cracks
P4-10.5-2	10.5	4	4145	123,700	1464	0.003	730	1280	—	732	Yielding of reinforcement; small transverse crack at loaded end
P4-14-1	14	4	3790	108,200	961	0.008	475	670	850	480	Yielding of reinforcement; no cracks
P4-14-2	14	4	3790	108,200	961	0.004	470	830	—	480	Same as above
P4-17-1	17	4	4565	123,700	1008	0.000	425	—	—	504	Yielding of reinforcement; no cracks
P4-17-2	17	4	4565	123,700	1004	0.000	475	—	—	502	Same as above
P8-7-1	7	8	3235	34,300	1228	0.02	675	560	705	614	Bond; small longitudinal crack extending entire length of specimen on four faces; two small transverse cracks at loaded end
P8-7-2	7	8	3235	33,500	1214	0.02	710	585	750	607	Same as above
P8-14-1	14	8	4300	76,600	1371	0.03	540	565	730	686	Bond; small longitudinal crack extending entire length of specimen on two opposite faces; small transverse crack at loaded end
P8-14-2	14	8	4300	78,000	1394	0.02	595	835	1065	697	Same as above
P8-21-1	21	8	4865	105,300	1257	*	495	1055	—	628	Bond; small longitudinal crack extending entire length of specimen on two opposite faces; small transverse crack at loaded end
P8-21-2	21	8	4865	104,200	1243	0.03	395	970	1195	622	Same as above
P8-28-1	28	8	4015	108,000	966	0.01	250	705	840	483	Yielding of reinforcement; small longitudinal crack extending one-third length of specimen on two opposite faces; small transverse crack at loaded end
P8-28-2	28	8	4015	108,000	966	0.01	270	655	790	483	Same as above
P8-34-1	34	8	4060	123,800	912	0.001	335	—	—	456	Yielding of reinforcement; small longitudinal crack extending a short distance on two opposite faces; small transverse crack at loaded end
P8-34-2	34	8	4060	122,500	902	0.001	280	—	—	451	Same as above

*A large slip developed, but was not recorded.

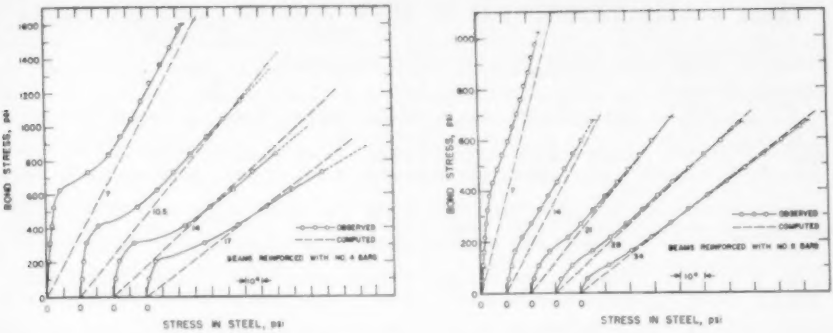

Fig. 6—Relationship between the computed bond stresses and both the average observed and computed steel stresses. Bond stress was computed from the formula

$$u = \frac{f_s A_s}{\Sigma_0 L}$$

where f_s is the stress in the bar computed by straight line theory. Numbers on curves indicate the length of embedment in inches

at the maximum load, but the failure was classified as bond failure because of the large slip at the free end of the bar.

The same criterion was applied in classifying the modes of failure of pull-out specimens. For the #4 bars, only the pull-outs with an embedment of 7 in. failed in bond, while the rest of the #4 bars failed by yielding. For the #8 bars, pull-outs with embedments of 7, 14, and 21 in. failed in bond, while those with embedments of 28 and 34 in. failed by yielding of the bars. This observation is in marked contrast with the results obtained with beams containing #8 bars. All of these beams failed by bond and developed large slips at the free ends.

Table 3 presents the values of u_m, the maximum value of bond stress, computed by the formula

$$u = \frac{f_s A_s}{\Sigma_0 L}$$

where L is the length of the shear span (which is the same as the length of embedment), and the other terms are standard. It is noted that f_s in this expression is the stress computed by straight line theory.

The values of u_m were adjusted to take into account the variation of the compressive strength of concrete, f_c'. The adjusted values of u_m were obtained by multiplying the observed values by the factor $\overline{f_c'}/f_c'$, where $\overline{f_c'}$ is the average value obtained for each group of beam specimens containing bars of a given size. The adjusted values of u_m are also given in Table 3. In view of the comparatively narrow range of values of f_c' in each group of beams, the values of u_m adjusted as described did not

*A large slip developed, but was not recorded.

TABLE 5 — LOADED-END SLIP IN BEAM AND PULL-OUT SPECIMENS

Beam No.	Length of embedment, in.	Loaded-end slip, in., in reinforcing bar based on f_s, psi of								
		20,000	30,000	40,000	50,000	60,000	70,000	80,000	90,000	100,000
4-7-1	7	0.0001	0.0002	0.0018	0.0042	0.0066	0.0092	0.0132	—	—
4-7-2	7	0.0001	0.0002	0.0013	0.0042	0.0068	0.0098	0.0142	—	—
4-10.5-1	10.5	0.0001	0.0002	0.0018	0.0045	0.0069	0.0098	0.0131	0.0175	0.0232
4-10.5-2	10.5	0.0000	0.0004	0.0020	0.0042	0.0064	0.0093	0.0122	0.0164	0.0224
4-10.5-3	10.5	0.0000	0.0003	0.0016	0.0036	0.0060	0.0090	0.0124	0.0172	0.0228
4-14-1	14	0.0000	0.0005	0.0022	0.0050	0.0070	0.0105	0.0134	0.0169	0.0204
4-14-2	14	0.0002	0.0004	0.0023	0.0058	0.0082	0.0120	0.0173*	0.0257*	—
4-17-1	17	0.0002	0.0006	0.0026	0.0050	0.0072	0.0093	0.0119	0.0140	0.0152*
4-17-2	17	0.0003	0.0008	0.0026	0.0050	0.0076	0.0105	0.0148	0.0185	0.0215
Avg slip		0.0001	0.0004	0.0020	0.0046	0.0070	0.0099	0.0132	0.0168	0.0221
8-7-1	7	0.0029	—	—	—	—	—	—	—	—
8-14-1	14	0.0022	0.0050	—	—	—	—	—	—	—
8-14-2	14	0.0023	0.0054	0.0103*	—	—	—	—	—	—
8-21-1	21	0.0019	0.0044	0.0057	0.0074	0.0092	—	—	—	—
8-21-2	21	0.0019	0.0049	0.0050	0.0078	—	—	—	—	—
8-28-1	28	0.0021	0.0044	0.0058	0.0073	0.0090	0.0108	—	—	—
8-28-2	28	0.0026	0.0048	0.0064	0.0081	0.0102	—	—	—	—
8-34-1	34	0.0021	0.0037	0.0046	0.0058	0.0073	0.0086	0.0103	—	—
8-34-2	34	0.0023	0.0045	0.0061	0.0081	0.0098	0.0120	0.0139	—	—
Avg slip		0.0023	0.0046	0.0056	0.0074	0.0091	0.0105	0.0121		
Pull-out No.										
P4-7-1	7	0.0011	0.0041	0.0084*	0.0136*	0.0196*	0.0278*	0.0375*	—	—
P4-7-2	7	0.0032	0.0054	0.0082*	0.0122*	0.0169*	0.0216*	0.0280*	—	—
P4-10.5-1	10.5	0.0016	0.0030	0.0046	0.0064	0.0085	0.0106	0.0129	0.0156	0.0184
P4-10.5-2	10.5	0.0016	0.0030	0.0048	0.0068	0.0090	0.0114	0.0139	0.0167	0.0199
P4-14-1	14	0.0019	0.0037	0.0059	0.0088	0.0124	0.0164	0.0206	0.0256	0.0299
P4-14-2	14	0.0028	0.0047	0.0066	0.0093	0.0121	0.0158	0.0187	0.0226	0.0272
P4-17-1	17	0.0016	0.0036	0.0057	0.0080	0.0104	0.0128	0.0156	0.0187	0.0224
P4-17-2	17	0.0018	0.0032	0.0049	0.0069	0.0093	0.0118	0.0146	0.0117	0.0214
Avg slip		0.0020	0.0038	0.0054	0.0077	0.0103	0.0131	0.0160	0.0195	0.0232
P8-7-1	7	0.0113*	0.0236*	—	—	—	—	—	—	—
P8-7-2	7	0.0089*	0.0241*	—	—	—	—	—	—	—
P8-14-1	14	0.0038	0.0099	0.0179	0.0273	0.0381	0.0513*	—	—	—
P8-14-2	14	0.0042	0.0085	0.0134	0.0193	0.0273	0.0404	—	—	—
P8-21-1	21	0.0029	0.0055	0.0092	0.0151	0.0199	0.0265	0.0340	0.0427	0.0557
P8-21-2	21	0.0043	0.0085	0.0131	0.0181	0.0237	0.0301	0.0373	0.0436	0.0477
P8-28-1	28	0.0054	0.0117	0.0199	0.0294	0.0352	0.0379	0.0387	0.0388	0.0346
P8-28-2	28	0.0049	0.0100	0.0169	0.0251	0.0333	0.0376	0.0391	—	—
P8-34-1	34	0.0032	0.0063	0.0101	0.0150	0.0209	0.0273	0.0330	0.0379	0.0394
P8-34-2	34	0.0036	0.0070	0.0109	0.0159	0.0221	0.0283	0.0334	0.0342	—
Avg slip		0.0040	0.0084	0.0139	0.0206	0.0276	0.326	0.0359	0.0394	0.0444

*These values were not included in the average slip.

Note: The loaded-end slip values in the beam specimens represent the average for both ends of a beam.

differ materially from values adjusted by using the ratio of square roots of compressive strengths.

Fig. 6 shows the relationships between the computed bond stresses and the corresponding computed and observed steel stresses. It can be seen that the curves representing these relationships approach each other as the load increases, thus indicating that the computed average bond stresses approach their actual average values at higher loads.

The bond strengths u_m given in Table 3 correspond to a nearly complete state of collapse of the beam specimens shown in the views of typical beams after test in Fig. 5. Before this state of collapse was attained, the beam specimens reached a condition which must be regarded as one indicating an impending bond failure. An examination of the bond-stress-slip relationships shown in Fig. 7 indicates that in a number of cases an impending bond failure was associated with either a free-end slip or a loaded-end slip which lay in a relatively narrow range of values of slip. This fact was taken into account in devising criteria for defining the "critical" value of bond stress.

An examination of Fig. 7 shows that there is no assurance that the loaded-end slip by itself is sufficient to indicate an impending bond failure. This is shown in the case of those beams in which the loaded-end slip became almost static while the free-end slip continued to increase with load. This situation may arise where a wide crack develops near the support causing larger slips at the free end of the bar than at its loaded end. This relationship is demonstrated graphically for Beams 8-21-1, 8-21-2, 8-28-2, 8-34-1, and 8-34-2 in Fig. 7. Therefore it was decided to adopt two criteria for defining critical bond stresses. The "critical" bond stress is defined as being the lesser of the bond stresses corresponding to either a free-end slip of 0.002 in. or a loaded-end slip of 0.01 in. It is noted that slip measurements were made at both ends of a beam, and the critical bond stress was always taken as corresponding to that end of the beam where the governing slip developed first.

The values of bond stresses corresponding to a free-end slip of 0.002 in. and a loaded-end slip of 0.01 in. are given in Table 3, along with the values of critical bond stresses. A comparison of these stresses shows that in one beam with a #4 bar, and in four beams with #8 bars, the critical bond stress was governed by the free-end slip.

Although it might appear that different values of loaded-end slip should be used for defining critical bond stresses in beams with different lengths of embedment, the data in Table 5 and graphs in Fig. 8 indicate that loaded-end slip was essentially independent of the length of embedment for a given size bar. Thus it can be seen that the loaded-end slip for a given size bar is determined primarily by the stress in the bar. For example, at a stress of 60,000 psi the loaded-end slip was about 0.007 in. for #4 bars and 0.009 in. for the five beams containing #8 bars

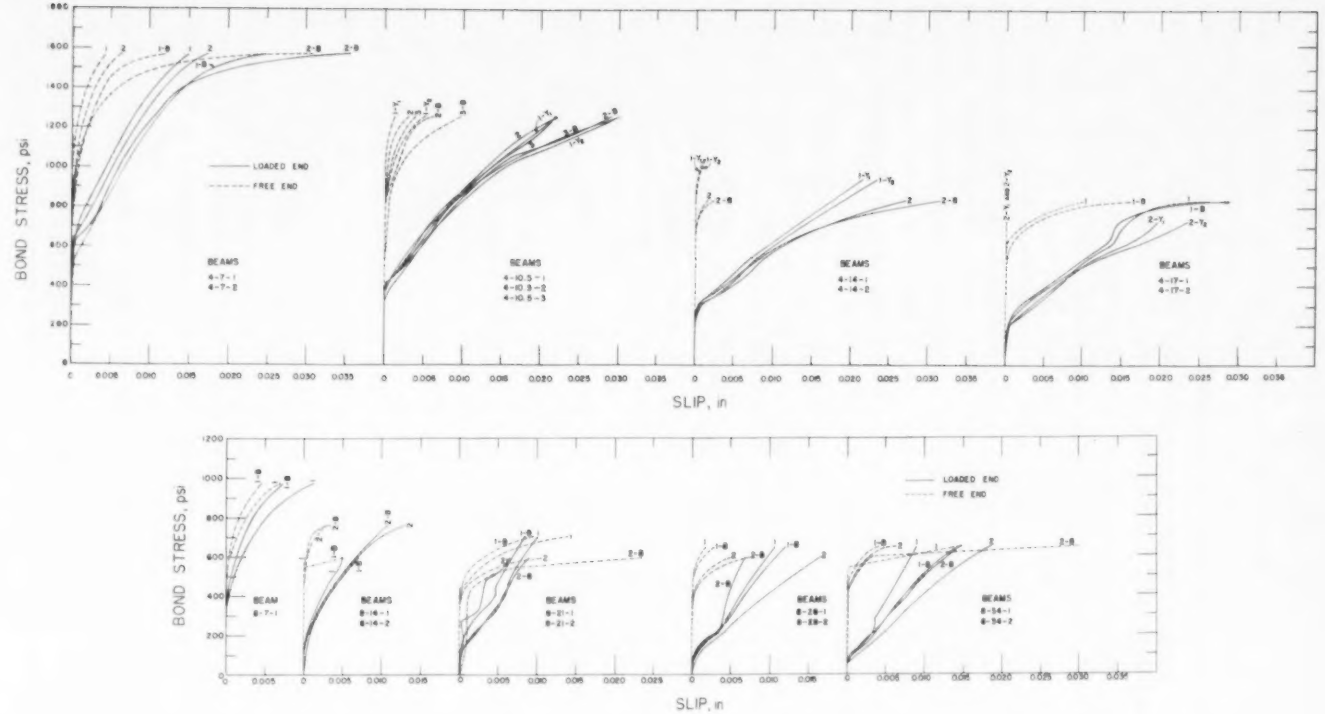

Fig. 7—Computed bond stress in beams versus loaded and free end slip. Numbers 1 to 3 on the curves identify the beams from a group of companion specimens. The end of the beam failing in bond is designated by B, and yielding of the reinforcing bar is denoted by Y. The subscripts following Y identify the ends of a given beam

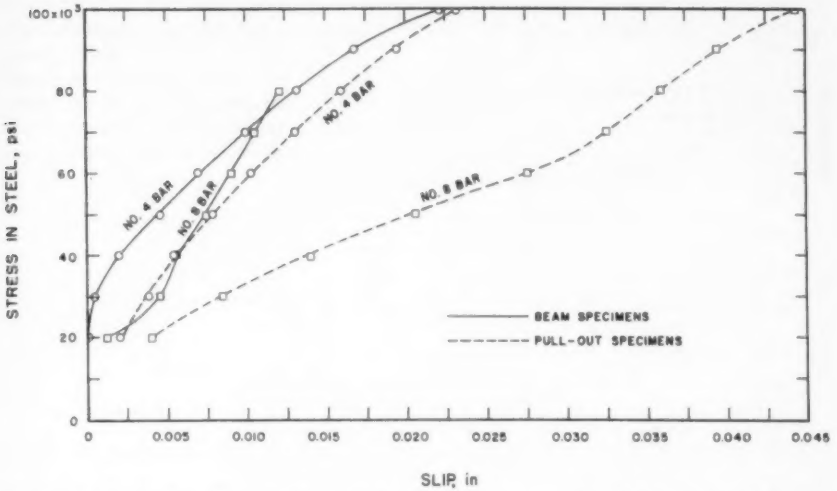

Fig. 8—Computed stress in the reinforcing bar versus loaded-end slip. The slip was obtained by averaging the values for all lengths of embedment at a given steel stress (see Table 5). For beam specimens, both ends of the beam were included in the computations

which had not failed at that stress. The stresses at a loaded-end slip of 0.01 in. which was the value selected for defining the critical bond stress by one of the criteria, were about 70,000 and 66,000 psi in #4 and #8 bars, respectively. It is noted that the data in Table 5 and in Fig. 8 are based on the loaded-end slips averaged for both ends of a beam. It should also be pointed out that the larger values of slip did not always occur at the end of the beam failing in bond.

Fig. 9 shows clearly that both u_m and u_c decrease with an increasing L/D for a given bar size. The critical bond stress u_c decreases from 1170 to 530 psi for the #4 bar, and from 650 to 470 psi for the #8 bar, as L/D increases from 14 to 34. It may be noted that the values of critical bond stress for the two size bars approach each other as L/D increases.

It may be of interest to note the relationships between the values of f_s corresponding to u_c [designated as $(f_s)_c$] and the L/D ratios shown in Fig. 10 for #4 and #8 bars. These data show that it is possible to develop a steel stress of about 70,000 psi with a 14-diameter embedment of #4 bars whereas a 34-diameter embedment is required with #8 bars to develop an average stress of about 66,000 psi. It must be emphasized that these relationships are based on the *critical* bond stress u_c, rather than the maximum value, u_m.

An examination of the data given in Table 5 shows that the values of average loaded-end slip for both beams and pull-outs were in general

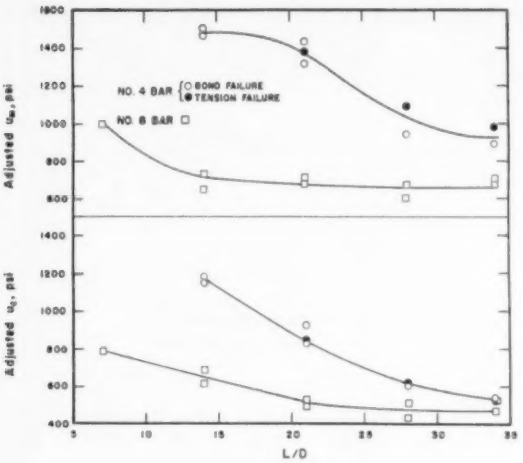

Fig. 9—Adjusted bond strength, u_m, and the adjusted critical bond stress, u_c, versus the ratio of length of embedment to diameter of bar, L/D. These data apply only to the beam specimens

based on a progressively decreasing number of specimens as the stress increased. Thus, for beams with #4 bars, the number of specimens varied from nine at a stress of 20,000 psi to five at a stress of 100,000 psi. For beams with #8 bars, the number of specimens varied from nine at 20,000 psi to two at 80,000 psi. The number of beam specimens decreased in this manner because of the progressively larger number of bond failures which developed as the steel stresses increased.

Fig. 8 shows the stress-loaded-end slip relationship for both beam and pull-out specimens. It is significant that greater loaded-end slips were observed in pull-out specimens than in beams for the same size of bar at a given stress in steel, particularly for the #8 bar. This observed difference in slips reflects primarily the fact that pull-outs are free of transverse cracks and therefore the entire extension of the embedded bar is registered as a loaded-end slip in pull-outs, while in beams a portion of the slip between concrete and steel appears at cracks within the shear span.

The critical values of bond stress, u_c, presented in Table 3 were also adjusted in the same manner as were the bond strengths, u_m, to take into account the variation of the compressive strength of concrete. Fig. 9 shows the adjusted values of u_m and u_c plotted against the L/D ratio.

The relationship between the critical bond stress u_c and the ratio L/D shown in Fig. 9 is an indication that within the scope of this study the bond strength of a beam is a function of the stress in the tensile reinforcement as well as of the shear force at the support since $4L/D$ equals f_s/u. However, the relationship as given in Fig. 9 does not lend itself in that form as an aid to designers of reinforced concrete members, particularly in the case of members subjected to a uniformly distributed

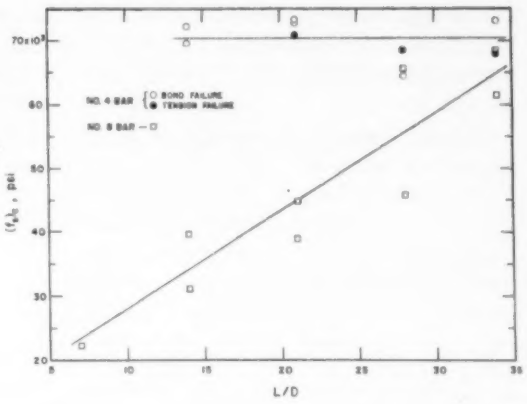

Fig. 10 — Relationship between the steel stresses corresponding to the critical bond stresses u_c, and the ratios of length of embedment to diameter of bar, L/D

load. To increase the usefulness of the information presented in Fig. 9, the value of u_c was plotted against the ratio $u_c/(f_s)_c$ in Fig. 11, where $(f_s)_c$ is the computed maximum steel stress in the beam specimen corresponding to u_c. The values of u_c plotted as the ordinates were adjusted to take into account the minor variations in the compressive strength of concrete, whereas their values in the ratio $u_c/(f_s)_c$ were not. The values of $(f_s)_c$ are given in Table 3.

Fig. 11 shows that a linear relationship appeared to exist between the critical bond stresses u_c and the ratio $u_c/(f_s)_c$ for #4 bars. For the #8 bars, this relationship was less certain, but a linear relationship was assumed as a basis for calculation. The large difference between the values of u_c for these two bar sizes at the higher values of $u_c/(f_s)_c$ reflects a similar large difference between values of u_c for the lower values of L/D shown in Fig. 9. As the values of $u_c/(f_s)_c$ decrease corresponding to longer lengths of embedment, the values of u_c for the two bar sizes approach one another (within the scope of this investigation).

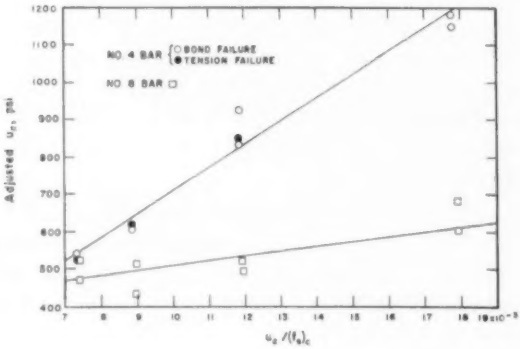

Fig. 11 — Adjusted critical bond stress, u_c versus the ratio $u_c/(f_s)_c$ for beams [$(f_s)_c$ is the computed stress in the reinforcing bar]. The ratio of u/f_s is equal to $D/4L$ for this series of beam tests

The relationship between u_c and $u_c/(f_s)_e$ shown in Fig. 11 was established on the basis of simply supported beam specimens tested under concentrated loads, and it remains to be established that this relationship is also valid for beams under uniformly distributed loads.

The mathematical expressions for the straight lines plotted in Fig. 11 for beams with #4 and #8 bars are respectively:

$$\text{adjusted } u_c = 62{,}600 \frac{u_c}{(f_s)_e} + 85 \quad \ldots\ldots (1)$$

$$\text{adjusted } u_c = 13{,}000 \frac{u_c}{(f_s)_e} + 380 \quad \ldots\ldots (2)$$

It may be of interest to compare the values from beam tests of $u_m/2$ and $u_c/2$ with the maximum allowable bond stress of 350 psi given in the ACI 318-56.[4] For all beams with #4 bars $u_m/2$ was greater than 350 psi, while for six out of the nine beams with #8 bars $u_m/2$ failed to attain 350 psi. The values of $u_c/2$ were less than 350 psi for all #8 bars, except the bar embedded 7 in., while for #4 bars, four out of nine were less than 350 psi.

The greater disparity between pull-out and beam bond strength which was observed for the #8 bars is due to the fact that all beams with #8 bars developed cracks within the shear span, whereas beams with #4 bars developed these cracks only for a length of embedment of 17 in. Mains[5] pointed out that pull-out and beam specimens yield comparable results only when the length of the pull-out is comparable to the distance from the free end of the bar in a beam to the nearest crack within the shear span. Thus, the absence of cracks in most of the beams with the #4 bars made them more nearly comparable to the pull-outs with the same lengths of embedment.

SUMMARY

Bond strengths were determined with 18 beam and 18 pull-out specimens containing deformed reinforcing bars with a nominal yield strength of 100,000 psi. Two sizes of bars were used, #4 and #8, and the lengths of embedment ranged from 7 to 17 in. for #4 bars and from 7 to 34 in. for #8 bars. The beams were cast with the tensile reinforcement near the bottom. The test procedure with the beam specimens was essentially that recommended by ACI Committee 208, Bond Stress.

The following observations were made:

1. The bond strength was found to decrease with increases in the length of embedment for a bar of a given size. The bond value also decreased with an increase in the bar diameter for a given length-diameter ratio.

2. The ultimate bond stresses in the pull-out specimens agreed in general with the values obtained in beams with #4 bars. However, for #8 bars the bond strengths in pull-outs were significantly greater than the values obtained with beams.

3. The loaded-end slip in the beam and pull-out specimens increased with the stress in the bar and was essentially independent of the length of embedment for a given size bar. However, the loaded-end-slip relationships were different for the beam and pull-out specimens.

4. Two criteria were used to define "critical" bond stresses: a loaded-end-slip of 0.01 in. and a free-end slip of 0.002 in., whichever occurred first. There was no assurance that the loaded-end slip alone would indicate impending bond failure in a beam.

5. A linear relationship appeared to exist between the critical bond stresses and the ratio of bar diameter to length of embedment, D/L, for the #4 bars. For the #8 bars this relationship was less certain, but a linear relationship was assumed to exist.

Inasmuch as the quantity $D/4L$ is equal to u/f_s (ratio of bond stress to maximum steel stress in a simply supported beam subjected to concentrated loads), it is suggested that the relationship between the critical bond stress and u/f_s may be used as a basis for selecting the proper bond stress as a function both of the shearing force at the end of the beam and the maximum stress in the reinforcement.

6. For all beams with #4 bars $u_m/2$ was greater than 350 psi (maximum allowable bond stress of ACI 318-56), while for six out of the nine beams with #8 bars $u_m/2$ failed to attain 350 psi. The values of $u_c/2$ were less than 350 psi for all #8 bars except the bar embedded 7 in., while for #4 bars, four out of nine were less than 350 psi.

ACKNOWLEDGMENT

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Title No. 57-51

A Case of Abnormally Slow Hardening Concrete for Tunnel Lining

By LEWIS H. TUTHILL, ROBERT F. ADAMS, SHELLY N. BAILEY, and RONALD W. SMITH

When job concrete does not set for several days there is usually a reason. When the correct reason is not found, the blame may fall where it is undeserved but worse still, nothing is learned to prevent a recurrence. This paper describes such an experience and the related testing which shows that one cause of belated hardening, when a lignin-base water-reducing admixture is used, is too little SO_3 in the cement.

■ WHEN 2 TO 4 DAYS ELAPSE before concrete hardens appreciably beyond initial set, it may, for all practical purposes, be considered abnormally slow in hardening. Retardation of several hours or half a day beyond normal may be purposefully induced for certain valid reasons in construction, but retardation for several days is abnormal and its unexpected occurrence can be most disturbing on a construction operation geared to safe removal of forms in 10 to 12 hr as in tunnel lining.

Heretofore, when concrete was slow in hardening, and if no admixture was used, it was usually due to:

(1) Concrete that was placed cold and stayed cold, although not necessarily frozen,

(2) Malicious or inadvertent inclusion of a considerable quantity of sugar, molasses, or other organic material, or

(3) Possibly some irregularity of the cement.

If an admixture capable of retardation was used, and concrete was much slower in hardening than usual, it was usually considered due to:

(1) A gross overdosage of several times the normal amount of the admixture, or

(2) Crudity and variability of the product such that at times the admixture consisted of a material with an exceptionally high content of sugar or other very retardant ingredient.

There is no record in such a case, of the very slow hardening being ascribed to, or actually identified with, a sudden change in some aspect of the cement, even though it was well within the limits of ASTM requirements.

It is the purpose of this paper to describe in detail a series of events, circumstances, and tests which are considered conclusive evidence that the grossly slow hardening which occurred with use of a lignin-base water-reducing retarder was due to a sudden reduction of the percentage of SO_3 in the cement from about 2.0 to about 1.6. In view of this evidence it is suggested that at least in some past cases of seriously delayed hardening, such deviation from cement uniformity, though within specification limits, could well have been the cause rather than the suspected overdosage or crudity of the lignin admixture.

JOB CONDITIONS

This experience occurred during lining of the arch and sidewalls of Tunnel No. 5, an 8800-ft railroad tunnel on relocation of the Western Pacific Railroad around the reservoir to be formed by Oroville Dam of the California Feather River Project. The lined tunnel is 25 ft 11 in. high above the paved invert carrying ballast and track. The sides are vertical, rising to a semicircular arch of 9-ft radius. The excavation is 100 percent supported with 8-in., 35-lb steel H-sets spaced 2 to 6 ft apart and with such blocking as the ground seemed to require. Concrete lining is required to be at least 8 in. thick between the face of these steel sets and the forms. Minimum thickness to rock is 18 in. Overbreak results in an average thickness of about 34 in. The concrete is reinforced transversely with #9 bars at 9-in. centers 2½ in. clear from the forms, and longitudinally with #5 longitudinal bars at 12-in. centers placed back of the transverse steel.

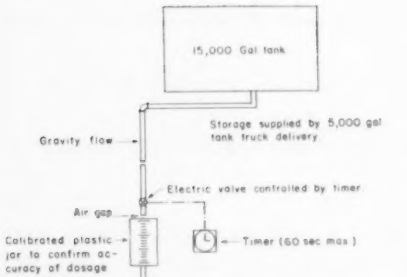
15,000 Gal tank
Storage supplied by 5,000 gal tank truck delivery
Gravity flow
Electric valve controlled by timer
Air gap
Timer (60 sec max.)
Calibrated plastic jar to confirm accuracy of dosage for each batch. Size is ample for ½ maximum dosage
Hand operated (open and close) valve
Valve adjusted to distribute admixture throughout water for each batch
Batch water to mixer

Fig. 1 — Dispensing facilities for water-reducing retarding solution, Tunnels 4 and 5, Feather River Project

Concrete materials were automatically batched in 1.5-cu yd units and five of these units were combined in a 7.5-cu yd batch in each of two truck mixers mounted with their discharges toward the center on a standard railroad flatcar. Three cars were thus equipped. Cement was weighed separately. Aggregates were weighed cumulatively. The water-reducing retarder was batched as indicated in Fig. 1, twice for each 7½-cu yd batch because a visual container large enough for the required amount in one dosage was not then available. The rate of discharge was adjusted so these two half-batch dispensings flowed most of the time the metered water was

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Ronald W. Smith was employed 2 years by the California Department of Water Resources as an engineering aid. He is also a civil engineering student at Sacramento State College and was recently employed by Aerojet General Corp. in their hydraulics testing laboratory.

flowing into the mixer. Each second half-dosage was not retained for check in the container but flowed directly through it. It is important, in view of what happened, to note (1) that the accuracy of the time valve measuring the admixture was checked every other time as the first half-dose for each 7½-cu yd batch was verified in the container, and (2) there was not time enough during the batching of the 7½-cu yd load to permit more than the two half-dosages to flow, the way the discharge control valve was set.

At the forms, concrete was delivered on a belt conveyor to a 7½-cu yd supply hopper which fed concrete to the two 3-cu yd remix hoppers of two dual concrete pumps. These placed the concrete through two 8-in. I. D. slick lines into the crown of the arch at a combined average rate of 75-cu yd per hr. From there it flowed downward and forward into the sidewalls. There it was vibrated, as best concrete on a slope can be vibrated, with immersion-type vibrators working through the three rows of doors with centers 8 ft apart horizontally on each side of the forms. Concrete as delivered in the arch was kept moving laterally by an occasional slug of air through the slick lines and was consolidated as the arch filled by means of form vibrators.

Concrete consistency was indicated by an average 5½-in. slump as mixed. This slump was required because with slump loss ranging up to 1 in., batches of much lower slump could result in concrete in the forms with a slump less than 4 in. In tunnel lining, concrete of such lower slumps invites a hazard of poor consolidation and incomplete filling. Excessively high slumps were avoided because the contractor was impelled to work with some margin within a 7-in. positive rejection limit which was strictly imposed. With the slump variation inherent in a truck-mixing operation, he dared not crowd the 7-in. limit as mixed,

for fear of exceeding it and having the concrete rejected. When slump of concrete as mixed approached this limit, inspectors required reductions in water added at the plant; however, the contractor was responsible for all additions of water to the mix.

The concrete mix contained 5 sacks of Type II Cement DI per cu yd. Sand passing No. 4 mesh was 33.5 percent of the total aggregate by solid volume. This is a clean, well-graded, light colored, largely quartz and quartzite sand that has been reworked from placer mining. Its specific gravity is 2.63. The coarse aggregate is a largely dark colored gravel composed of amphibole schists, meta-andesite, meta-basalts, granites, and other largely igneous rocks reclaimed from gold dredge tailings in the Feather River flood plain near Oroville. Its specific gravity is 2.91 and it was graded as follows in the concrete mix: No. 4 to ⅝ in., 28 percent; ⅝ to 1¼ in., 33 percent; and 1¼ to 2½ in., 39 percent. Water requirement of the arch concrete at 5½-in. slump as mixed was 240 lb. per cu yd for a W/C of 0.51. Two and one-half percent air was entrained by means of a resin agent. This was the least air which would noticeably improve workability and plasticity in the 2½-in. aggregate concrete; more would tend to reduce strength. For the 7½-cu yd batch only about 12 oz of a half-pound-to-to-the-gallon solution was required in addition to the water-reducing admixture. For each 7½-cu yd batch, 285 oz of the 40 percent water-reducing Admixture Solution QA were required to provide ¼ lb of sodium lignosulfonate solids per sack of cement. This solution was supplied in 5000-gal. tank-truck lots and stored on the job in a 15,000-gal. tank from which it flowed by gravity to the batching plant (Fig. 1).

A full day's run of 96 ft of arch and sidewall concrete required as much as 600 cu yd of concrete. Job storage of aggregate was sufficient for two such days of operation. The 900 bbl of cement in storage at the batch plant was only enough for 720 cu yd of concrete. Both were supplied by bulk truck and trailer shipments from their sources. Coarse aggregate was finish screened and washed on a 4 x 14-ft, horizontally operating, three-deck, vibrating screen with seven spray bars, which was mounted over and delivered directly to the batch plant bins. The rewashing was necessitated by mishandling of the material with a bulldozer when building stockpiles. Because of the well-paced operation, this late wetting did not cause as much slump variation as expected and helped to keep concrete temperature down in hot weather.

Cement deliveries were accepted on certification that it complied with specifications and the first delivery from each new mill lot was accompanied by the mill tests on which the certification was based. All cement delivered to the project, including this which was involved in abnormally slow hardening, fully met all specification requirements including freedom from false set. This was confirmed in tests by the California Department of Water Resources, of random samples taken from each lot of cement on the job.

WHAT HAPPENED

Wednesday, Jan. 27, 1960, concrete was placed as usual in two 48-ft lengths of arch and sidewall, one each beginning at Station 1188+04 and 1156+46. The first was completed about noon. About 10 p.m., after the usual 10-hr interval after placing the last concrete in the crown, the forms were lowered from the first of these two sections. From an area some 16 ft square in the crown, the concrete between the forms and the reinforcement came down with the forms (Fig. 2). Inspection then and next day indicated that in this area the concrete had stopped hardening at the initial set while adjacent concrete in the same placement had hardened normally. The next day and for 4 days the affected concrete remained the same. It could be molded into a ball. A cylinder was molded from it Thursday afternoon, January 28. Saturday it was still possible to rub cement out of the mortar onto the fingers. Monday morning the beginning of hardening was evident.

Thursday, January 28, when this delayed set was first apparent to only a limited extent, it was believed less likely to be due to the cement than to some malfunctioning of batching. Another 48-ft length of arch and sidewall was placed from Station 1187+54 with the same mix including the water-reducing Admixture QA. Batching was carefully watched. That night most of the concrete in this arch back to the steel came down with the forms, including a short section at the end which came down from behind the steel, bending it down where it had been lapped in the crown (as shown in Figure 3).

As a result of this second failure to set, concrete placed Friday from Station 1187+04 contained no admixture and an additional ½ sack of cement per cu yd. This hardened sufficiently for satisfactory form stripping but inspectors felt that it was colder and less hard than had been normal for its age. Nevertheless, from the successful stripping of this placement, it was inferred that some incompatibility existed between the current mill run of cement and the admixture. After further examination of the admixture supply and batching procedure previously described, it was felt certain the admixture could not have changed in character or dosage; cement was the only

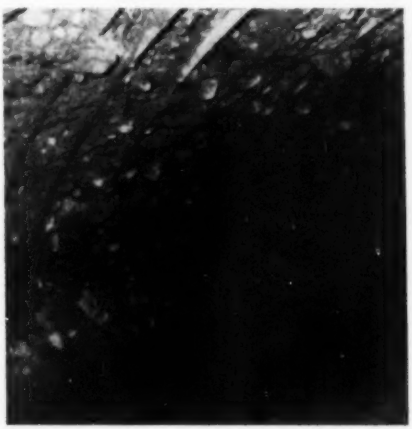

Fig. 2 — Unhardened concrete between reinforcing and forms came down when forms were lowered at regular time, 10 hr after forms were filled

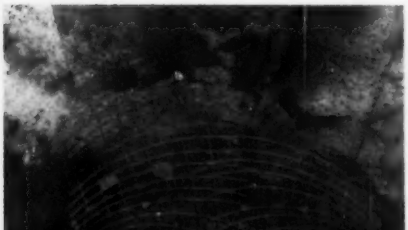

Fig. 3 — Some lapped reinforcing at crown of arch was bent in as unhardened concrete came down when forms were lowered at usual interval after placing concrete

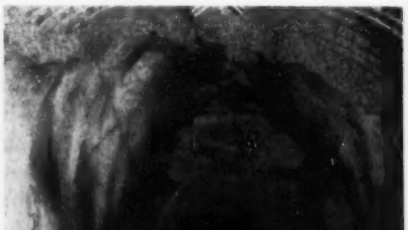

Fig. 4 — Light and dark areas in this and Fig. 3 indicate variation in hardening. Light areas were warm and hard; dark areas were cold and unhardened

active ingredient which could be changing in some respect and evidently was doing so.

In an examination Saturday of the sections placed Wednesday and Thursday which had not set normally, it was noted that parts were light colored and other areas, including the areas where some concrete came down with the forms, were dark in color (Fig. 3 and 4). The light areas were dry, hard and normal; the dark areas were damp, and barely hardened. This clearly indicated sudden variation in the cement, as this brand of Type II cement had been in uneventful and satisfactory use since Oct. 13, 1959. A recent summary had shown that it had produced 5 percent higher strength than the Type I cement of the same brand used prior to October 13.

During the weekend, having lost faith in this lot of cement and not wishing to abandon use of the admixture which he felt eased concrete pumping and minimized cold joints, the contractor arranged to exchange cement remaining in job storage for cement from a different mill run. This exchange was accomplished Monday night, February 1.

Also, it so happened, a 5000-gal. delivery of the liquid Admixture QA, ordered prior to these occurrences, arrived Monday and was placed in the storage tank. Gravity, appearance, and rate of hardening tests of mortar showed that it was the same as previous supplies (Fig. 5 and 8). The Department of Water Resources was also reluctant to abandon use of the water-reducing admixture.[2-5] It was serving admirably to offset to some extent the higher slumps that practical considerations usually dictate in a tunnel-lining operation. Water reduction appeared to average 8.4 percent and a recent summary had shown 13 percent

better strength at 28 days in concrete of the same cement content and slump. Moreover, it was felt that the admixture had contributed to a uniformity of strength that was unusual for tunnel lining concrete since a *V* of 8 percent was obtained for 55 tests of arch concrete in Tunnel 4.

With the new cement and admixture, concrete of the regular mix including the admixture was placed Tuesday, February 2, in a section from Station 1186+54. That night it had set sufficiently to strip satisfactorily. Inspectors noted nothing unusual in its temperature or hardness. It was thought the trouble was past and we might never know what had caused it. At that time all chemical tests had not been completed on samples of the previous lot of cement.

Wednesday, February 3, the same mix was placed in the section from Station 1186+04. That night the full 48 ft of arch concrete back to the reinforcing cage came down with the forms. The contrary performance Tuesday and Wednesday, if nothing else, proved again the lack of uniformity of the cement; nothing else had changed. The contractor was advised to return to the Type I cement or to change to another brand of Type II. He chose the latter and operations returned to normal.

SAMPLING AND TESTING

The first tests made were the standard Gillmore time of set tests on Cement 60C28 sampled Thursday, January 28, when evidence of slow hardening was first apparent. Information on the various samples of cement mentioned is summarized in Table 1. As shown in the following, the first cements sampled behaved normally in time of set tests both with and without Admixture QA added to the neat cement in the job mix proportion of ¼ lb per sack:

Cement		Time of set, hr:min	
		Initial	Final
		2:20	4:15
60C28	Thursday, January 28, cement only	2:00	4:20
60C28	Thursday, January 28, with admixture	2:00	5:00
60C30	Friday, January 29, cement only	2:00	4:30

Two other samples of cement taken Monday, February 1, exhibited normal times of set without admixture (although the initial set was slightly faster than usual), as shown in the following data. With ¼ lb per sack of Admixture QA, rapid stiffening was evident at 10 min, the initial set was accelerated and the final set was retarded slightly. With ½ lb per sack QA, rapid stiffening was again evident at 10 min, and both initial and final sets were accelerated. These rates of setting, varying as they did from those above, were indicative of the lack of uniformity of the cement performance when the admixture was used.

Cement	Amount of admixture	Time of set, hr:min		Rapid stiffening
		Initial	Final	
60C-46 (February 1, from cement returned to mill)	No admixture	1:35	4:20	None
	1/4 lb per sack QA	0:45	5:25	At 10 min
	1/2 lb per sack QA	0:25	1:55	At 10 min
60C-47 (February 1, from cement returned to mill)	No admixture	1:30	4:00	None
	1/4 lb per sack QA	1:15	4:45	At 10 min
	1/2 lb per sack QA	0:35	1:35	At 10 min

The standard time of set tests showed nothing unusual in the performance of the cement by itself, but with the admixture included it sometimes behaved abnormally and exhibited acceleration rather than retardation. These tests did not identify anything about the cement which might be the cause of abnormally slow-hardening concrete.

However, rate of hardening tests based on resistance of mortars to penetration of Proctor needles at increasing intervals after mixing, similar to ASTM C 403, showed unusually slow and abnormal hardening (Fig. 5) and gave measurable physical evidence of the degree of this abnormality. Using in such tests only batches of mortar similar to the mortar in job concrete, with different cements and admixtures, conclusive identifying results were obtained.

For these mortars predampened job sand was used at the same ratio of cement to sand as in job concrete. The mortar was mixed as outlined in ASTM C 305, except that a small part of the mix water was withheld for final consistency adjustment, and a large Hobart mixer, the 13-qt, Model N-200, was used. Consistency of the mortar was judged by eye

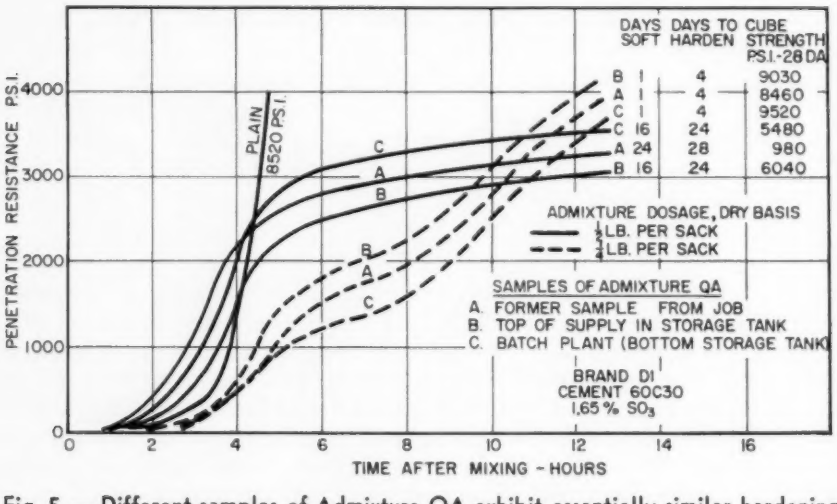

Fig. 5 — Different samples of Admixture QA exhibit essentially similar hardening effect with same dosage and same cement in concrete mortar

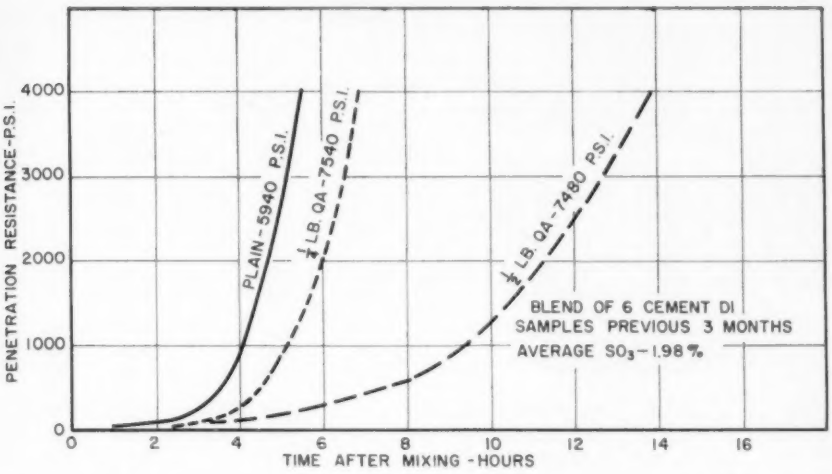

Fig. 6 — Concrete mortars with previous job cement set normally with regular and double dosage of lignin Admixture QA from batch plant

with the aid of several checks with the flow table showing the flow of the mortar between 80 and 90 percent. This consistency was considered fairly representative of the mortar fraction of job concrete. The mortar was placed in 6 x 9-in. cans, where it remained under wet burlap in laboratory air at 70-75 F during the intervals between penetration tests.

When the rate of hardening was abnormally slow, the mortar specimens were placed in the fog room after about 24 hr where the progress of hardening was observed periodically. After 28 days 2-in. cubes were cut from many of these specimens. This is the strength shown on many of the rate of hardening curves in the figures.

Despite circumstances favorable to uniformity of character and dosage of the liquid admixture, it was at first suspected as somehow causing the slow hardening. Accordingly a sample was obtained from the top of the solution in the supply tank to compare with a sample from the batch plant which draws directly from the bottom of the tank. These two samples, together with a sample in the laboratory from a previous delivery, were each used in hardening test mortars with Cement 60C30 obtained at the batching plant at the close of placing Friday, January 29. Fig. 5 shows that all three samples of the admixture performed similarly in the mortar, which was extremely slow hardening when either ¼- or ½-lb per sack dosages were used, although it set normally without the admixture.

To determine whether the cement or the admixture had changed, remains of six earlier samples of the cement were located and blended for the test shown in Fig. 6. Only the plant sample of admixture was

TABLE 1 — PHYSICAL AND CHEMICA

Laboratory No.	Brand and type	Sample	Physic					
			Normal consistency, percent	Time of set Gillmore, hr:min		Autoclave expansion, percent	Compressi 2-in. cub	
				Initial	Final		3 days	7 days
60C-28	DI-II	From job 1/28, Thursday	23.6	2:00	4:20	+0.128	1640	2690
60C-30	DI-II	From job 1/29, Friday	23.6	2:00	4:30	+0.130	1830	2880
60C-55	DI-II	From job 2/3, Wednesday	23.4	1:50	4:20	+0.126	1800	3000
60C-59B	DI-II	From job 2/4, Thursday	—	—	—	—	—	—
60C-60	DI-II	From job 2/4, Thursday	23.6	2:40	4:15	+0.107	1890	2970
60C-75	DI-II	New bin	23.6	3:35	4:35	+0.047	2170	3930
Blend, 6 samples cement used 10/13/59 to 1/4/60	DI-II	Highest SO_3	23.6	2:40	5:00	+0.111	1950	3110
	DI-II	Lowest SO_3	23.4	2:00	3:15	+0.028	1650	2700
	DI-II	Average	23.5	2:18	4:27	+0.052	1802	2846
58C-94	DI-I	Old laboratory sample	24.0	2:00	3:40	+0.037	2000	3000
58C-154	AC-II	Old laboratory sample	23.4	2:25	3:55	+0.033	2000	3010
60C-57	AC-II	From job 2/5, Friday	24.0	3:35	5:15	+0.057	1850	2870
57C-360	EP-II	Old laboratory sample	22.6	2:30	5:40	−0.033	1760	2430

used and, in contrast to its performance in Fig. 5 with current cement, rate of hardening was normal. Moreover, as shown in Fig. 7 two other brands of Type II cement hardened normally in mortars with ¼ lb per sack dosages of the admixture sampled at the batch plant.

Since a new 5000-gal. delivery of admixture arrived Monday, February 1, it was necessary to be sure this would perform no differently than the several hundred gallons remaining in the tank the previous week. Fig. 8 shows rate of hardening results with the new supply of admixture used with cement used the previous week, 60C30, and with cement from the new mill run from which delivery started Monday night, February 1. Sample 60C55 of the latter was taken at the batch plant during concrete placing operations Wednesday morning, February 3. It will be recalled that this is the last concrete which did not harden normally.

Hardening was abnormally slow with both cements at dosages of both ¼ and ½ lb per sack. The similarity of the delayed hardening with Cement 60C30 in Fig. 8 and in Fig. 5 and its satisfactory performance with other lots of Cement DI and other brands (Fig. 7, 13, and 14) shows that the new supply of admixture was the same as before and that the difficulty was not due to some change in the admixture. Results in Fig. 8 also showed that the new lot of Cement 60C55 hardened no faster than the cement it replaced. It will be recalled that some concrete placed Wednesday morning, February 3, came down with the forms. Only variation in the cement could explain why concrete placed Tuesday, February 2, did not come down.

PROPERTIES OF PORTLAND CEMENTS

properties						Chemical analysis, percent					
strength psi	Air entrainment, mortar, percent	Fineness, air permeability, sq cm per g	False set—Federal penetration, mm			C_3S	C_2S	C_3A	C_4AF	Alkalies Na_2O equivalent	SO_3
28 days			Initial	Final	Difference						
5000	8.27	3260	38	30	8	45	28	6	14	0.49	1.54
4950	9.73	3101	37	32	5	46	27	6	14	0.47	1.65
5110	9.07	3067	37	30	7	48	25	6	14	0.44	1.58
—	—	—	—	—	—	48	25	7	14	0.53	1.73
4960	9.84	3144	36	28	8	50	23	7	14	0.53	1.72
4500	10.00	3270	33	18	15	51	22	7	13	0.45	2.53
4900	10.8	3338			12	51	32	7	16	0.70	2.11
4350	8.5	2976			1	40	23	5	13	0.38	1.76
4697	9.9	3126			6	47	26	6.5	14	0.57	1.98
3780	10.40	3060	37	34	3	47	26	11	8	0.94	2.40
4490	6.72	3040	36	27	9	54	26	5	8	0.47	1.73
4150	8.40	2920	33	28	5	5	28	6	8	0.61	1.72
3970	11.00	3540	37	1	36	52	30	3	8	0.21	1.98

It will also be noted in Fig. 8 that when mortars contained a normal and double dosage of a non-lignin admixture with Cement 60C30, the rate of hardening was normal.

At this stage of operations and testing, sufficient chemical analyses of the cements involved became available so the data could be examined in search of something different from the six cements which set at a nor-

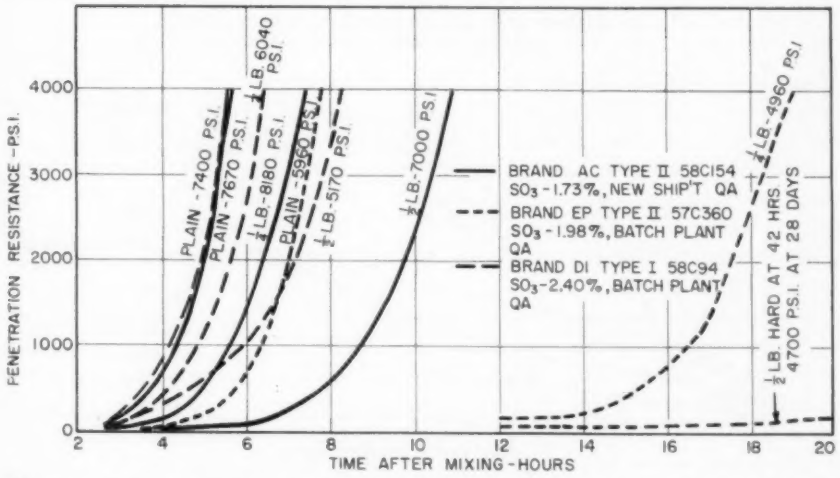
PENETRATION RESISTANCE - P.S.I.
TIME AFTER MIXING - HOURS
PLAIN - 7400 P.S.I.
PLAIN - 7670 P.S.I.
½ LB. 6040 P.S.I.
½ LB. 8180 P.S.I.
PLAIN - 5960 P.S.I.
½ LB. 5170 P.S.I.
½ LB. 7000 P.S.I.
BRAND AC TYPE II 58C154
SO3 - 1.73%, NEW SHIP'T QA
BRAND EP TYPE II 57C360
SO3 - 1.98%, BATCH PLANT QA
BRAND DI TYPE I 58C94
SO3 - 2.40%, BATCH PLANT QA
½ LB. 4960 P.S.I.
½ LB. HARD AT 42 HRS
4700 P.S.I. AT 28 DAYS

Fig. 7 — Concrete mortars with other brands of cement hardened normally with job lignin admixture. Although one was slow with a double dosage, the hardening characteristics were normal

mal rate with the admixture as indicated in Fig. 6 and in 3½ months' work with them in the tunnel. The only consistent difference was in the percent of SO_3. In the six earlier samples the percent of SO_3 ranged from 1.78 to 2.11, averaging 1.98 percent. In Sample 60C30 it was 1.65 and in 60C55 it was 1.58 percent. In Cement 60C28, the first cement sample obtained when difficulty was encountered, the SO_3 was 1.54 percent. Pertinent chemical and physical data on the cements are shown in Table 1.

Whether these were significant differences could only be determined by further tests of these cements after adding sufficient powdered

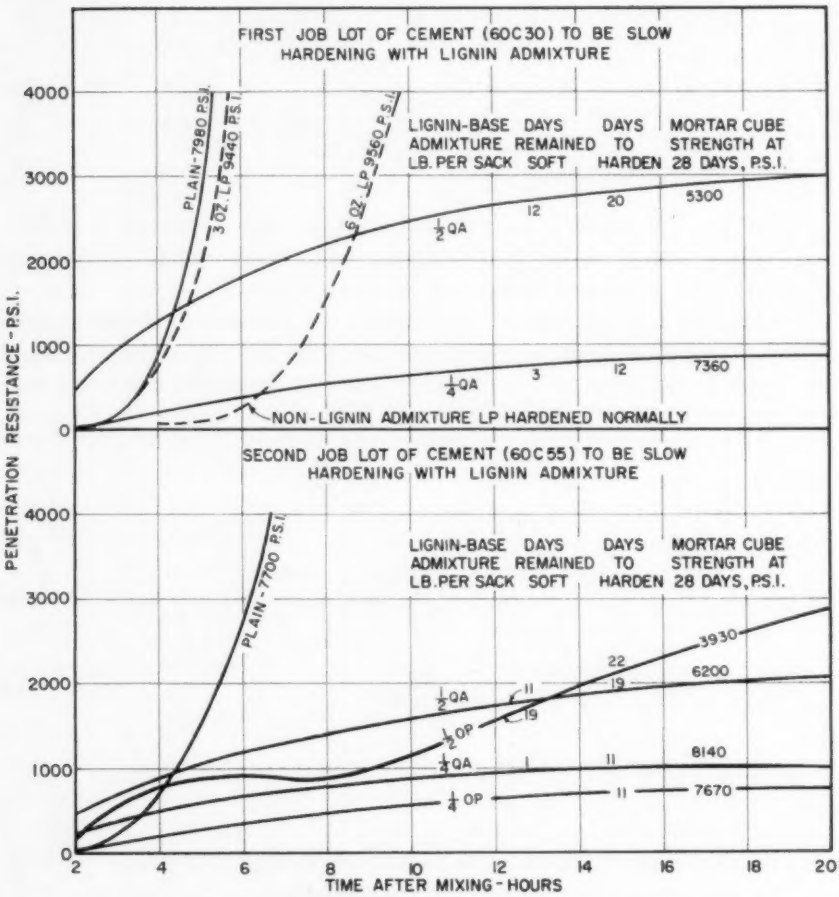

Fig. 8 — Both mill lots of Cement DI at job hardened abnormally slow with new delivery of job Admixture QA and another lignin-base Admixture OP. Cement 60C30 with new QA is similarly slow hardening as it was with previous QA supply shown in Fig. 5. Performance was normal with non-lignin Admixture LP

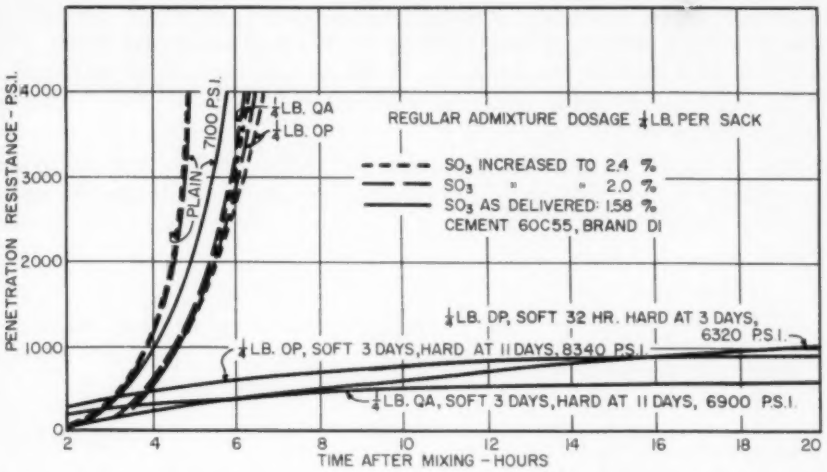

Fig. 9—When SO_3 is increased to 2.0 or 2.4 percent by addition of gypsum, job Cement 60C55 hardens normally with lignin-base Admixtures QA and OP. With SO_3 at 1.58 percent as delivered, hardening is very slow with regular ¼-lb dose of lignin-base Admixtures QA, OP, and DP

gypsum to bring the SO_3 up to 2.0 percent or more. In all these remaining tests reported the Admixture QA was from the latest delivery.

Fig. 9 shows that by increasing the 1.58 percent SO_3 of Cement 60C55 to 2.0 and 2.4 percent, extremely slow hardening with the lignin-type admixtures is changed to entirely normal hardening with the two of them tested, one being the job admixture. In all cases the dosage was ¼ lb per sack.

Fig. 10 similarly shows the effect of increasing the 1.65 percent SO_3 of Cement 60C30 to 2.0 and 2.4 percent. In addition to the effect with ¼-lb dosages, the effect with ½-lb dosages was included with the job admixture and one other proprietary lignin-base admixture. Here again the additional gypsum changed the mortars from very slow hardening to normal hardening, even with the double dosage of the water-reducing retardant admixtures, except that an increase to only 2.0 percent SO_3 did not seem quite enough to prevent a slow set with the ½ lb double dosage of the job admixture, although this dosage hardened properly with 2.4 percent of SO_3. Moreover, with the higher percent of SO_3, none of the combinations exhibited the rapid stiffening to initial set or more, with very slow subsequent hardening, that was often shown, especially with the larger dosage, as in Fig. 5, 8, and 10.

From the evidence in Fig. 9 and 10 there appears to be no doubt that if enough gypsum is used to get the percent of SO_3 well above 2.0 that hardening will be normal at least with the regular ¼-lb dosage of a lignin admixture. However, tests with two samples of cement taken at

different times Thursday, February 4, exhibited different hardening characteristics although their percent of SO_3 was about the same, 1.72 which took 6 days to harden and 1.73 which was hard in 10 hr. Apparently some other factor than the amount of SO_3 is involved in these results shown in Fig. 11.

Fig. 12 shows another example, similar to the one in Fig. 10, of an increase to 2.1 percent of SO_3 not being enough to make mortar with Cement 60C55 harden at a proper rate when it contained a ½ lb double

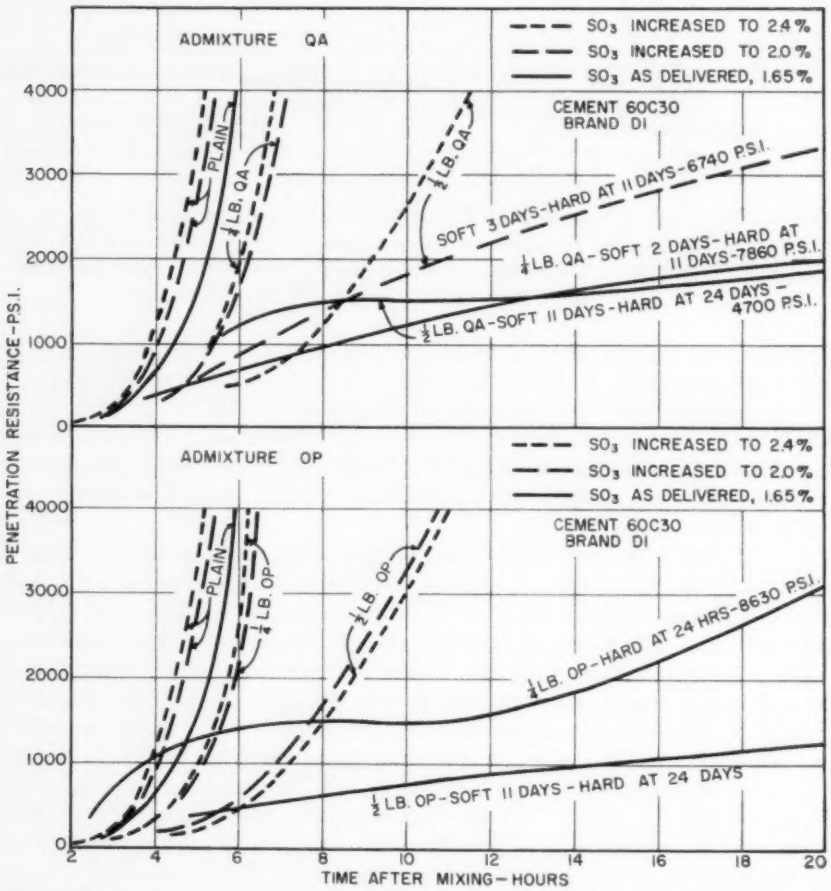

Fig. 10—When SO_3 is increased to 2.0 or 2.4 percent by addition of gypsum, job Cement 60C30 hardens normally with regular ¼ lb, and in most cases with double ½ lb dosage of lignin-base Admixture QA (upper) and OP (lower). With SO_3 at 1.65 percent as delivered, hardening is very slow with regular ¼-lb dose per sack of these admixtures

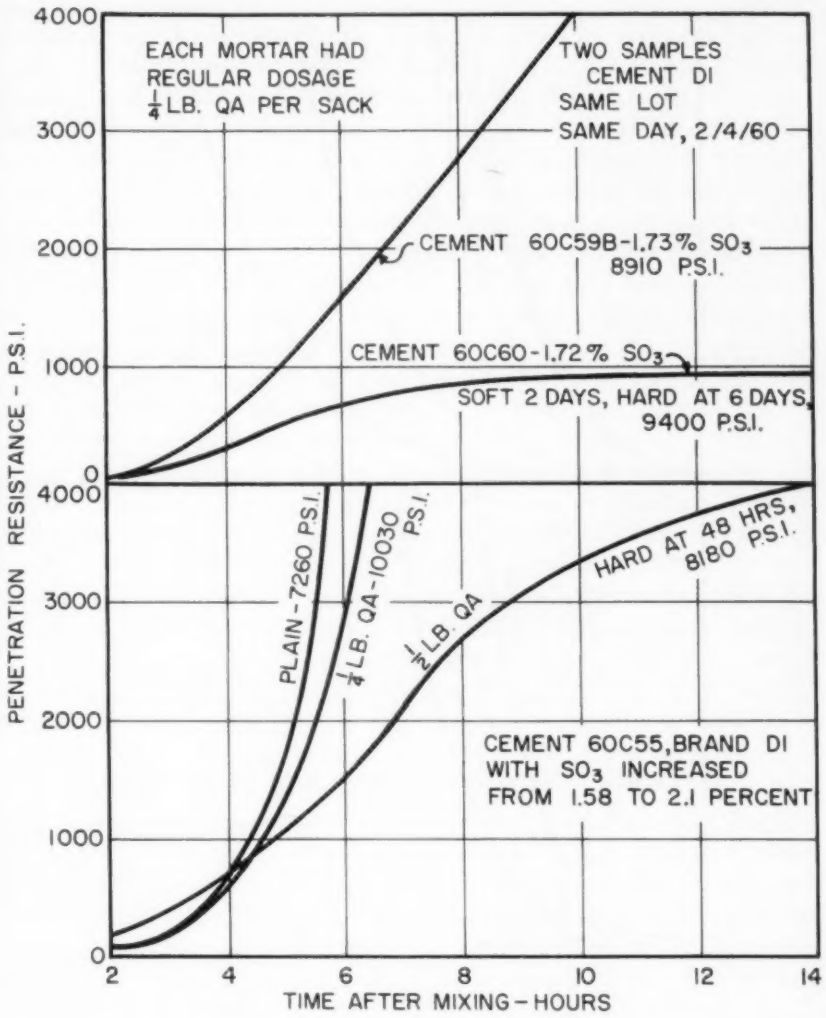

Fig. 11 (top) — Different effects of similar percent SO_3 indicates this may be critical SO_3 content, particularly if other elements of the cement vary. Fig. 12 (bottom) — When SO_3 in Cement 60C55 is increased from 1.58 to 2.1 percent, hardening is normal for regular ¼-lb dose of lignin-base Admixture QA, but somewhat delayed with a double ½-lb dose. Fig. 10 shows greater delay for the ½-lb dose with 2.0 percent SO_3 (only 0.1 percent less) in this cement

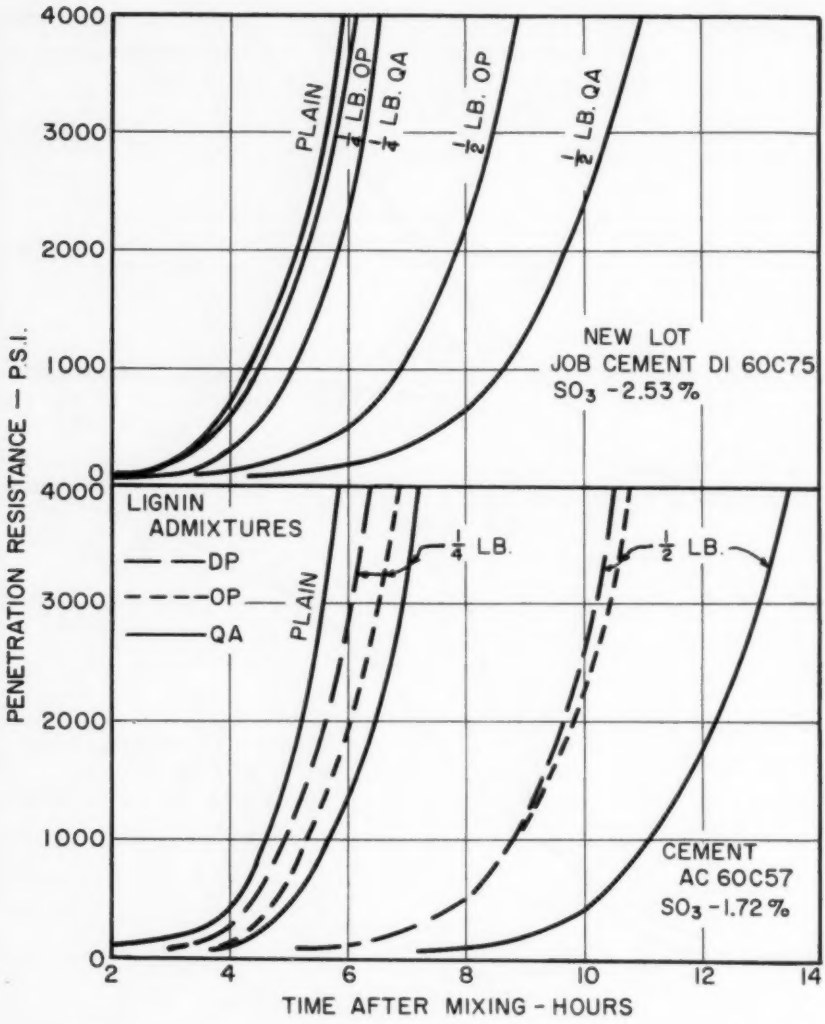

Fig. 13 (top) — When it contains 2.53 percent SO_3, job Cement DI exhibits normal hardening with lignin-base Admixture OP and QA in doses of ¼ and ½ lb per sack. Fig. 14 (bottom) — Cement AC with 1.72 percent SO_3 hardens normally with regular ¼ lb and double ½ lb per sack doses of lignin-base Admixtures DP, OP, and QA

dosage of the job admixture. Hardening was normal with the ¼-lb dosage which is recommended and has been satisfactory for ordinary cool temperatures above 60 F. However, in Fig. 6 it will be recalled that mortar with the ½-lb dosage hardened normally with the earlier job cement which had an average SO_3 content of 1.98 percent. Perhaps gypsum is more efficient if it is ground with the cement.

Fig. 13 shows the performance of the cement from a bin purposely ground with more gypsum, after the foregoing evidence of the importance of enough SO_3 was presented to the cement company. The percent of SO_3 was 2.53 for the sample used in the laboratory tests, although the bin average was 2.27 percent. It will be noted that this not only produced a mortar that hardened normally with ¼-lb dosages of two lignin-type admixtures, but that mortar containing the ½ lb double dosage hardened in a reasonable time also. The contractor returned to the use of Cement DI, with ample SO_3, on March 14 and subsequent results were good to the end of the job.

As previously mentioned, before this effect of the amount of SO_3 was learned, a different brand of Type II cement was substituted on the job. This was based on the tests shown in Fig. 7 with samples of other brands that were in sealed storage at the laboratory. Accordingly, as delivery of cement from the new source commenced, a sample was taken for a measure of its rate of hardening performance with the job and other lignin-based admixtures. Its content of SO_3 was 1.72 percent. The result is shown in Fig. 14. With all three admixtures the rate of hardening is normal, including mortars containing the ½ lb double dosage.

HOW GOOD IS THE SLOW-HARDENING CONCRETE?

All the affected concrete did not come down with the forms. For the most part the reinforcing cage provided enough added support to keep the remaining concrete from falling and with hardening somewhere between initial and final set, it was stiff enough to stand securely in the walls and for some distance above the spring line, without distortion. Doubts were raised as to the potential of this concrete for strength and some suggestions were heard that all of it should be removed.

Before the concrete placed January 27 and 28 had started to harden it was decided that there was no reason to remove it at once. If it got no harder it would still be easy to remove. If it did harden, the chances were it would develop adequate strength, since slow-hardening concretes are usually stronger ultimately than similar concretes with an ordinary rate of hardening. When it became evident Monday, February 1, that the unset concrete was going to harden, the only action permitted was

TABLE 2 — STRENGTH OF SLOW SETTING CONCRETE IN TUNNEL 5

Date concrete mixed	Date cylinders molded	Compressive strength, psi, 6 x 12-in. cylinders				Remarks
		7 days (1)	12 days	28 days (2)	91 days	
January 27	January 27	1870		3800		5 sacks
January 27	January 28				5750(1)	5 sacks
January 27	January 27	2190		3630		5 sacks
January 28	January 28	1850		4110		5 sacks
January 29	January 29	2220		4685		5½ sacks; no admixture
February 2	February 2	2190		4285		5 sacks
February 2	February 2	2110		3805		5-sacks
February 3	February 3	1960		3705		Cement 60C55
February 3	February 4		3890(2)	4660(3)	5620(2)	5 sacks
February 3	February 3	2060		3845		5 sacks
February 4	February 4	2500		4215		5½ sacks; no admixture
February 5	February 5	2520		3570		Different brand cement, 5 sacks

( ) Number of test cylinders.

to trim the damaged areas for proper repair. Repair was by shotcrete in the smaller shallow areas and by pumping 7½-sack, ¾-in. maximum concrete through a pin-valve attached to the forms where the damage was larger and deeper, but only after a month had elapsed and it was assured that remaining concrete had developed adequate strength. In the meantime the areas were water cured.

Only one test cylinder was molded January 28, from the unhardened concrete placed January 27. Facilities in the tunnel were unavailable January 29 to get unhardened concrete placed January 28 for test cylinders. However, when unhardened concrete placed February 3 was available February 4, the opportunity was taken to mold nine 6 x 12-in. cylinders. Also the regular set of test cylinders was made from each concrete as it was placed. None of the test cylinders, however, represented concrete in the arch as the test sample was taken from a batch when concrete only partly filled the sidewall forms. To what extent these represented concrete that was slow enough in setting to come down with the arch forms is uncertain except that the first test cylinders made February 3 were made about the same time, at 9 a.m., when cement Sample 60C55 was taken, which is shown to be slow hardening in Fig. 8 and 9.

The strengths of test cylinders shown in Table 2 are considered conclusive evidence of the strength developed in the concrete which remained of that placed January 27, January 28, and February 3. The remolded cylinders might be discounted possibly 10 percent for the benefit of reworking after partial setting had occurred. Even with this reduction they are ample to comply with a design strength requirement of 3000 psi at 28 days.

DISCUSSION AND CONCLUSIONS

The question naturally arises "Will hardening always be slow with lignin-base admixtures if the percent of SO_3 is low?" A Type V cement produced in another state and low in SO_3 is known to be thus slow in hardening. However, one large supplier of these admixtures states they have never previously identified extremely slow hardening with a low percent of SO_3 in the cement. Even though it may not always perform in this manner due to varying forms in which SO_3 is supplied and to the varying amounts of aluminates in different cements, the empirical evidence herein described indicates that it will be advisable hereafter, particularly with low-aluminate cements, to be sure in advance that the cement will perform satisfactorily with a lignin-base admixture.

Whatever the reason, there is no question but that the Type II cement in this case must have an SO_3 content well over 2 percent to be sure of avoiding grievously slow setting when lignin-base admixtures are used. The authors look forward to the possibility that discussion of this paper may develop an explanation for this.

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Title No. 57-52

Structural Models Evaluate Behavior of Concrete Dams

By JEROME M. RAPHAEL

An unusual arch and buttress dam was proposed as one possible design for the 750 x 5000 ft Oroville Dam. A plaster-celite, 1:200-scale model was cast in Fiberglas molds to duplicate the central portion of the dam. Live load tests were performed on the dam, simulating the uniformly increasing water pressure by a series of rubber bags under stepped pneumatic pressure. The control apparatus is described, and the results of the water load tests are given. A new method of dead load testing was devised by which the construction stresses as well as the dead load stresses of the completed dam could be determined. Dead load stresses are shown for various stages of construction, as well as the stresses under combined live and dead loads. The technique for dead load testing offers opportunity for checking the stresses in concrete dams under various proposed construction programs and should be especially useful for controlling the construction of the overhanging portions of doubly-curved arch dams.

■ ONE OF THE KEY STRUCTURES for the California Water Plan[1] is the Oroville Dam, to be built 7 miles above Oroville, Calif., where the Feather River emerges from the Sierra Nevada onto the plains of the Great Central Valley. Hydrologic requirements dictated a dam 750 ft above stream bed and 5000 ft long. Faced with the construction of a dam of this magnitude, the Department of Water Resources of the State of California gave consideration to five types of dams: a concrete gravity dam, a buttress dam, an arch and buttress dam, a multiple arch dam, and a rock-fill dam. Although of unusual height, none of these types presented unusual difficulty in analysis except the arch and buttress type, shown in the drawing in Fig. 1. For this unusual structure, the interaction of the various elements at the central arch structure was so complex that it was decided to construct and analyze a structural model of this section of the dam. This paper describes the methods used to determine the dead load and live load stresses on the model. While the model tests[2] demonstrated the feasibility of the arch-and-buttress design, the cost of foundation repairs for two weak areas removed all concrete designs from consideration, and attention is now (1960) being directed to a rock-fill design exclusively.

ARCH AND BUTTRESS DAM

The essential structural elements of the arch and buttress dam are shown in Fig. 2. The principal structural element is the central arch, which receives the water load on an inclined upstream face, and transmits this load on a curving catenary path to the foundations. A part of the water load is taken by the massive head of a buttress that transmits the load through the flank of the arch to a common foundation. The remainder of the dam is made up of independent massive head buttresses on 130-ft centers. At the deepest portion of the foundation, a mass concrete gravity plug acts as a temporary cofferdam during construction, and removes the highly stressed region at the vertex of the conical elements of the central arch. It can be seen that the central structure is highly indeterminate, since the riding buttresses not only transmit load to the arch in an indeterminate fashion, but they also so stiffen the arch that the distribution of stresses is made much more complex. Although each complete buttress may be considered to act independently, those in the immediate vicinity of the central arch affect the stresses in the arch through their influence on the deformation of the foundation. Hence, to perform a valid experimental analysis of the central arch, a number of adjacent buttresses had to be included in the loaded group to obtain comparable foundation deformations.

CONSTRUCTION OF THE MODEL

In a three-dimensional highly indeterminate structure such as the arch and buttress dam, it is essential that Poisson's ratio for the material of the model be the same as that for the material of the prototype, so that deformations and stresses in the model correspond to those of the prototype. This requirement narrowed the field of available model materials to cement mortars and plasters. Plaster diluted with diatom-

Fig. 1—Artist's drawing of the arch and buttress design for Oroville Dam; spillway at left, power plant under arch

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aceous earth or celite was chosen as the model material. This could be mixed easily to any desired strength and elasticity specification, cast to exact dimension, and worked easily if it were necessary to make modifications.

Similtude factors

Relationships among quantities measured on model and prototype are governed by the following similitude factors:

Scale $$\lambda = \frac{L_m}{L_p} \quad (1)$$

Loading $$g = \frac{\gamma_m}{\gamma_p} \quad (2)$$

Stress $$\frac{\sigma_m}{\sigma_p} = g\,\lambda \quad (3)$$

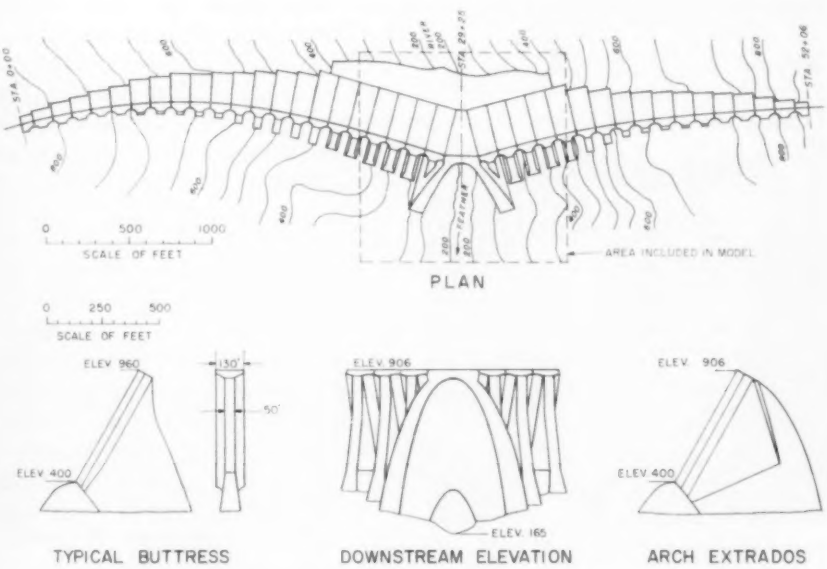

Fig. 2—Dimensioned views of arch and buttress dam

Strain $$\frac{e_m}{e_p} = g\,\lambda\,\frac{E_p}{E_m} \quad (4)$$

Deflection $$\frac{\Delta_m}{\Delta_p} = g\,\lambda^2\,\frac{E_p}{E_m} \quad (5)$$

L = a characteristic dimension
γ = density of solid or fluid loading
E = elastic modulus
m = model
p = prototype

As can be seen from Eq. (4), scale, material properties, and intensity of loading are interdependent. It was decided that as a first criterion, strain in model and prototype should be of the same order of magnitude. Since a number of successful model tests had been made using mercury as the loading medium, g was chosen as 13.6. Finally, it was decided that a scale of 1:200 would be as small as could be used and still maintain desired dimensional accuracy. Since it had already been assumed that the concrete of the dam would have an effective elastic modulus of 3.5×10^6 psi, the elastic modulus for the model was specified to be

$$E_m = 13.6 \times \frac{1}{200} \times 3.5 \times 10^6 = 240{,}000 \text{ psi}$$

Available schemes

A number of schemes are available for constructing plaster models. In some laboratories, notably the National Civil Engineering Laboratory,[3] Lisbon, oversized models are cast from which the final models are machined to dimension. To avoid laborious hand finishing, it was decided to cast the Oroville model to final shape. A combination of techniques were used for constructing the Oroville Dam model. The foundation was built up of prefabricated and dried plaster slabs, as shown in Fig. 3. The main arch, riding buttresses, and full buttresses were cast between Fiberglas molds, cured, and bonded to the foundation

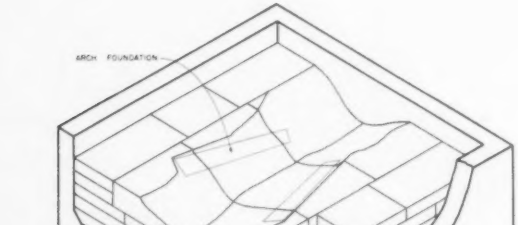

Fig. 3 — Isometric view showing method of construction of base of model

Fig. 4 — Constructing prototype by plastering between aluminum templates

Fig. 5—Checking fit of riding buttress prototype against arch

with an epoxy cement. The Fiberglas molds were made by laminating resin and Fiberglas to the surface of a prototype made of plaster screeded between a series of aluminum alloy templates as shown in Fig. 4, 5, and 6. The finished model, with some of the instrumentation in place, is shown in Fig. 7.

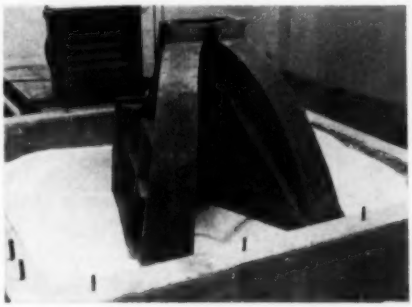

Fig. 6—Braced Fiberglas mold of main arch in place on completed model foundation

Materials

In constructing the various parts of the model, the modulus of elasticity of the gravity plug, of the various mass concrete structural elements, and of the foundation, was varied in the ratio of 1:2:4 in proportion to the modulus of elasticity of the corresponding parts of the prototype. This was accomplished by varying the proportions of water, plaster, and celite in the mixture while maintaining a workable consistency that enabled the mixture to fill the molds completely.

For any proportion of celite and plaster, the consistency of the mixture could be varied by successive additions of water from one too

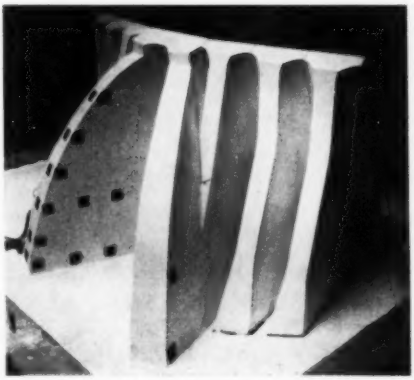

Fig. 7—View of completed model, with some of the strain gage rosettes in place

Fig. 8—Variation of elastic modulus with water-plaster and celite-plaster ratios for mixtures of uniform consistency

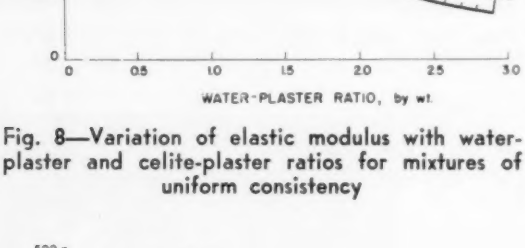

Fig. 9—Effect of drying on modulus of elasticity

stiff to fill the molds completely to one so thin that water segregated rapidly from the mixture. For these mixtures, the modulus of elasticity would correspondingly vary from a high value to a lower value with successive additions of water. By choosing only the mixture that had a desirable consistency, a curve such as Fig. 8 can be drawn for any given plaster, showing how the initial water-plaster ratio and celite-plaster ratio can be varied together to obtain hardened plasters over a wide range of moduli of elasticity while maintaining the desired initial consistency.

A second important water plaster relationship is shown in Fig. 9. The modulus of elasticity was determined for a number of cylinders from the same water-celite-plaster mixture after they were dried to varying degrees by a variety of treatments. It can be seen that despite the variation in the method of drying, the modulus of elasticity varied uniformly, as water was evaporated from the set plaster, to the point where all available free moisture had been removed. Beyond this point, elevated temperatures and continued drying removed part of the water of crystallization of the hydrated plaster, causing loss of strength. Thus to control this variable, all parts of the model were dried to zero free moisture content.

LIVE LOAD TESTS

A number of methods are available for simulating water load on a model dam. Water itself is rarely used to load a model, since the stresses resulting from water loading in the model are so low that strains will be hardly measurable. Hence mercury loading is often used to increase the density of loading and the resulting strains. Apart from the health hazard of working with mercury, there is one technical disadvantage. For any given head of fluid, there is no opportunity to check the linearity of behavior of the model with intensity of loading.

From the beginning of model testing, concentrated loads applied at the upstream face of the dam have been used to gage the linearity of strain in the model. In the Wilson and Gore experiments,[4] loads of varying amounts were applied using dead weights through distribution pads to the upstream face. For the three-dimensional case, multiple banks of hydraulic jacks are used (see Fig. 10), in the ISMES laboratory,[5] Bergamo, Italy. Concentrating the loads in this manner may present some difficulties in interpreting strains recorded on the upstream face of the model.

Replacement of the uniformly increasing hydrostatic load by a uniformly stepped load applied to the model by rubber tubes was first used by Pirtz[6] in his tests of a rock-fill dam in 1938. Pressure of each individual tube was controlled by varying the head of water in the tube. A constant linear relationship among the individual tubes was maintained by mounting the reservoirs for each tube on a boom, so arranged that by raising the boom, the intensity of loading could be varied at will. This principle was adopted for use in the Oroville Dam model, with the exception that water was ruled out as the loading medium because, as shown in Fig. 9 any leakage of water onto the surface of the model, in addition to short circuiting the strain gages, would modify the elastic properties of the plaster.

Loading device

In the Oroville Dam model tests, air was used as the loading medium. The air pressure in the individual load cells was controlled in a linearly increasing relationship by a loading device that is a derivation of Pirtz's hydraulic boom. The essential elements of the loading system are shown in Fig. 11. A number of closed water columns were connected in series by rubber tubes by which air could be

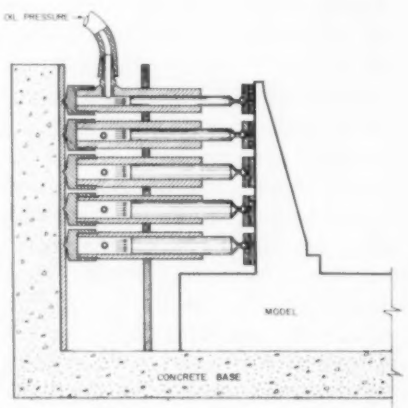

Fig. 10—Loading a model dam using multiple jacks

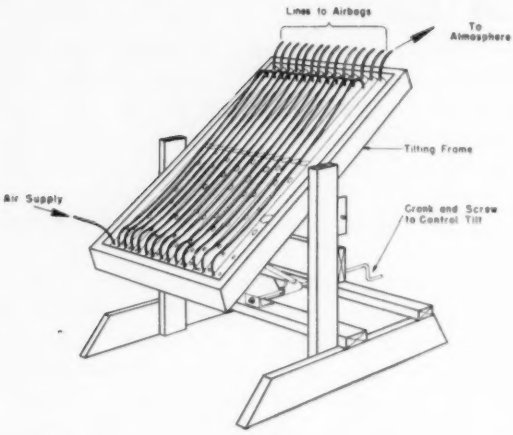

Fig. 11—Schematic diagram of live loading system for model

bubbled through each water column successively. The air pressure at the top of each water column differs from that of its neighboring column by the head of water in the column. The last water column in the series was maintained at half of the head of the adjacent water column. Thus a uniformly increasing pressure variation of 0, 1, 2, 3, 4 ... could be replaced by individual pressure bags having an average pressure of ½, 1½, 2½, 3½ ... giving the same average pressure variation. Since the pressure is dependent only on the hydraulic height of the water column, the intensity of pressure of the entire system could be varied at will by simply inclining the columns (see Fig. 12); when held flat, there is no pressure in the system; when held upright, there is maximum pressure in the system. Any variation in between could be adopted. Small rubber tubes were used to conduct pressurized air from the top of each water column to the individual pressure bags, and from there another tube led to a bank of mercury manometers so that the actual pressures on the model could be checked.

For most of the live load testing of the Oroville model the instrumentation proved so sensitive that a loading intensity of only 7.5 times that of water need be used. Under this loading, the resultant live load force passed close to the downstream edge of the base of the arch and outside to the base of the buttresses, thus developing high tensile stresses near the upstream face. To increase the stability of the model, a counterbalancing force was applied at the crest of the dam, thus effectively redirecting the resultant to the middle third of the base. This force was applied by spring loading an I-beam from which the load was transmitted to the model with a beam and roller grid. The actual counterweight force could be measured accurately by a pressure capsule. During testing, the counterweight force was applied first and maintained constant while the water load was applied and removed on the

dam. Thus, the strains recorded were only those due to the water loading. A view of the loading elements in place is shown in Fig. 13.

Live load stresses

Recording of strains on the surface of the model was made through the use of Baldwin-Southwark AR-1 wire wound, SR-4 strain gage rosettes. These were connected through a compensating board and a switchboard to a Datran resistance bridge indicator that automatically balanced all gage readings, recording directly the strain in microin. per in. on a display dial. This piece of apparatus was capable of extreme speed in reading, on several occasions producing 150 strain gage readings in 10 min.

The principal stresses resulting from the water load are shown for each strain gage rosette location in Fig. 14. This indicates that along the intrados, maximum compression of the order of 400 psi is found. The maximum tension on the intrados of 165 psi near the juncture of the arch and the plug would probably not appear in the prototype. It had been planned to break the structure between arch and plug by use of a mastic joint. Reasoning that the effect of dead load would be such that there would always be dead load compression across this mastic joint, and that any live load effects would appear as a diminution of the dead load stress already present, the model arch was actually bonded

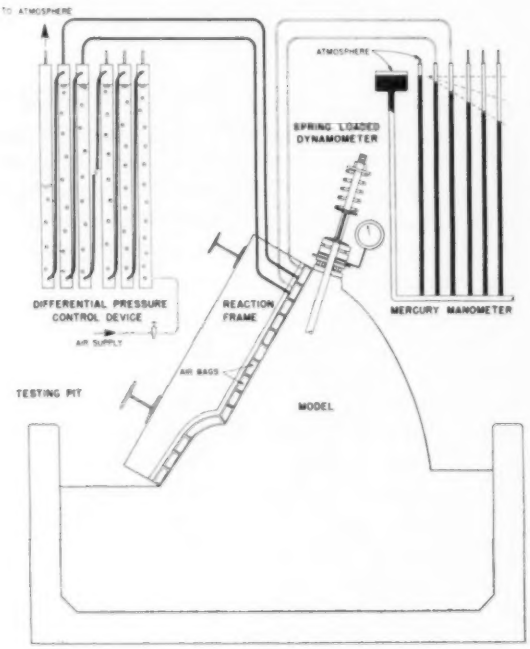

Fig. 12 — The tilting-table stepped pressure controller

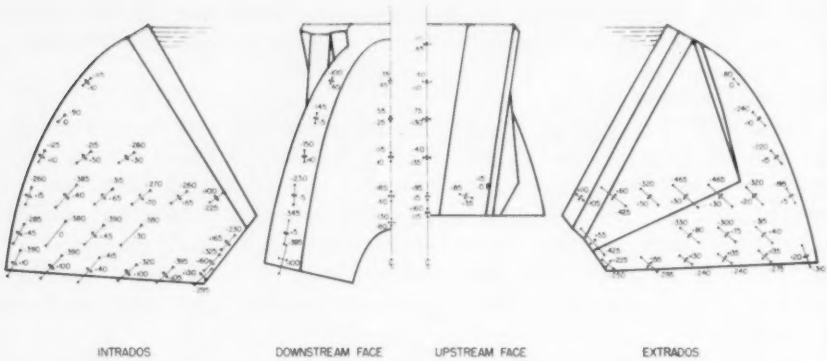

Fig. 13—General view of reaction frame and counter-loading device

to the plug at this location, thus allowing tensile stresses to appear under water load. For the extrados, high compressive stresses of 465 psi were found near the juncture of the riding buttress and the flank of the arch. Again, maximum tensile stresses were found along the base and at the juncture of the arch and plug. The principal stress directions were in general normal to the surface on which they were applied, except where modified by an external boundary that forms a principal stress surface. Since all these stresses are in addition to the existing dead load stresses, it becomes necessary to define the effect of dead load alone.

DEAD LOAD TESTS

In massive concrete structures such as dams, dead load stresses are quite likely to be the most important system of stresses. Studies of the actual stresses generated in dams during and on completion of construction have indicated that the maximum dead load stresses found on completion of construction have been only slightly modified by water load.[10]

A downstream view of the arch structure shows how much the flanks of the arch lean inward as construction progresses upward. Serious questions were raised whether a structure of this type would be stable during construction. Hence it became important to find a method for determining the dead load stresses, not only for the completed structure, but for various stages of construction. Dams are constructed in layers. At any stage of partial construction, resistance to deformation must be found in the parts of the incompleted structure actually present: any elements of strength to be found later in the completed structure are valueless during construction. Thus the emphasis on the determination of a scheme for the experimental analysis of the dead load stresses in the Oroville Dam was on methods that would give stresses accurately

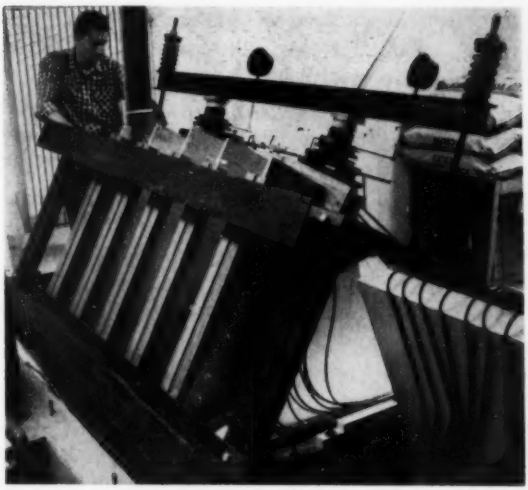

Fig. 14 — Distribution of principal stresses under water loading

for the partially completed structure as well as for the totally completed structure.

A number of schemes have been used successfully to determine dead load stresses in model dams. All of these were entirely successful for the problem at hand but were eliminated from consideration in testing the arch and buttress dam for the Oroville Dam. They are reviewed here, because they might be required in the dead load analysis of another type of dam.

Concentrated load method

The Wilson and Gore[4] experiment of 1908 simulated the body forces in a dam by applying concentrated loads to every part of the cross section of the gravity dam under test. This general scheme is still in use, in some laboratories[5] but was eliminated from consideration for Oroville because of the complex nature of the structure, which made it difficult to hang loads independently.

Centrifugal method

The centrifugal method is a second method for the experimental determination of dead load stresses in dams. The principle of the centrifugal method is the substitution of centrifugal forces for gravitational forces by rotating the model in a centrifuge. This method has been used with considerable success at the University of California by H. D. Eberhart[7] in a two-dimensional photoelastic model of the cross sections of several gravity-type dams. The model was made of a sheet of photoelastic material and rotated at high speeds at elevated temperatures (250 F.) for several hours until it reached thermal equilibrium. The temperature was then gradually reduced while maintaining the rotation constant until room temperature was reached. When the model was removed,

it had "frozen" within it a stress pattern resulting from the equivalent of an increased gravitational field. This could then be analyzed by photoelastic methods.

In another experiment, A. J. Durelli, Armour Research Foundation, went a step further toward three-dimensional analysis. His experiment[8] was concerned with the distribution of dead load stresses in a single buttress of a massive head buttress dam. In this experiment, a model of a single buttress together with part of its foundation was cast of a photoelastic material. As soon as this casting had solidified, it was transferred to a centrifuge and rotated rapidly while curing progressed. At the end of the curing period, the model was removed with the stresses frozen within it. The model was then cut into two-dimensional slices, and the stresses at various planes of the model were determined photoelastically.

Both of these tests were excellent solutions for the given essentially two-dimensional problem. After due consideration, however, certain limitations were found that rendered this method unattractive for the Oroville arch and buttress dam. Most plastic materials have a Poisson's ratio of 0.5 which is widely different from the 0.2 predicted for concrete of the dam. This meant that three-dimensional models made of plastics would not deform in all respects like the prototype. It follows that the stresses would not be distributed in a plastic model like those in the prototype. A second minor limitation was that, unlike gravity, centrifugal force is not constant throughout the model but varies as the square of the radius. Only by rotating a fairly small model on a long arm could the variation in the centrifugal field be held within feasible limits. A third limitation was that of size. Both centrifuges mentioned above were designed to handle fairly small models. Difficulties were anticipated in testing a model with the complexity of shape of the arch and buttress dam for Oroville in sizes that could be handled by these centrifuges. For larger plaster models, the problem of recording indications from surface strain gages on a model rotating at high speed in a centrifuge was difficult. These difficulties were enough to cause further search for the ideal dead load test method.

Immersion method

The method of immersion has been used in the Portuguese National Civil Engineering Laboratory for finding the stress distribution in a slice of a gravity dam. Biot[9] has shown the analogy between the two-dimensional state of stress resulting from body-force loading, such as the earth's gravitational pull on a structure, and the two-dimensional state of stress resulting from boundary loads. In this method an accurately cut section of the gravity dam together with a part of its foundation is inverted and lowered into a pool of mercury whose upper surface just touches the ground surface. Stresses at various points in the slice

are then related to corresponding stresses in the prototype dam by use of Biot's equations, which take into account the scale factor, and the relative densities of the immersing fluid and the material of the dam. While the method is valid for a two-dimensional analysis, a similar analogy has not been made for the three-dimensional state of stress found in the Oroville arch and buttress dam.

Inversion method

A promising method for the determination of dead load stresses in dams is the method of inversion.[3] In this method, a model of the structure is fabricated and fitted with strain gages. Zero readings are taken of the strain gages with the model in an upright position. The model is then inverted, hanging from its foundation, giving a change of loading twice gravity, and a second set of strain readings is taken. Strains in the model can be related directly to the stresses in the prototype through the use of similitude factors. Sensitivity of this test can be increased by decreasing the modulus of elasticity of the model material and by increasing its density. Plaster-celite mixtures with a modulus of elasticity of around 100,000 psi ordinarily weigh about 40 lb per cu ft. By substituting powdered barite for some of the celite in such a mixture, mixtures can be made having the same modulus of elasticity, but with a density almost three times as great. With a model material of this nature, and a 200-scale model of the Oroville Dam, it was considered that the distribution of gravity stresses could be obtained in the model with errors not greater than 3 to 5 percent. This method looked so attractive for the arch and buttress dam that design studies were developed for the fabrication of the apparatus. However, it proved impossible to design a workable apparatus for inverting this three-dimensional model without appreciable deflections in the foundation. These deflections produced enough twisting of the arch foundations to cause strains in the model many times those anticipated by gravity action. Hence this method was eliminated from further consideration.

Method of integration

The four methods for dead load testing described previously—applied loads, immersion, inversion, and centrifugal—had one important common defect. Field tests have shown the importance of the construction program in determining the distribution of stresses in a concrete dam. A completed model, loaded to full dead weight at the beginning of the test, does not reflect the incremental manner in which weight is added to the prototype structure and in which the structure gains its strength. Hence erroneous conclusions are reached concerning dead load stresses in the dam as construction proceeds. This defect was particularly bothersome for the model test of Oroville Dam because

questions had been raised regarding the behavior of the structure at intermediate stages of construction. Hence, a method was devised by which the stresses could be determined not only in the completed structure but also at various partial stages of completion following any given construction program. The method finally devised for the Oroville tests has been termed the "method of integration."

Since the dam is constructed in horizontal lifts, a model constructed in the same manner is stressed in its lower levels by material placed at higher elevations in a manner similar to the prototype, and only that part of the structure that has been constructed can take part in resisting the action of the imposed loads. If a similar method of construction were to be followed in the model, an awkward and time-consuming procedure would result. For each lift to be studied, molds would be assembled, plaster placed and dried, gages attached, the model tested, and the process would be repeated for as many times as the model was subdivided. Moreover, the response of the gages to the weight of model materials would be slight, and hence the accuracy of the method would be low.

Exactly the same results could be secured with much greater convenience by reversing the process: starting with the complete model, with all gages in place, and finding the effect of removing material in successive layers. The advantages are obvious: the existing model with all its instrumentation attached could be used at the close of the live load testing; there is no need to wait for curing of successive layers of plaster; the material will be homogeneous because it will all be placed at once. The one drawback mentioned above, that of the response of the gages to small changes of loads due to the low weight of the model materials, is eliminated by a slight change in testing procedure. As finally developed, in the method of integration for dead load stresses, the model is cut down by convenient increments, leaving a horizontal surface representing the top of a lift. On this surface, a uniform load is applied, of a magnitude great enough to cause a well-defined response in all the strain gages of the model. By using similitude factors, this is then related to the effect on the prototype dam of adding 1 ft of concrete at that elevation.

Thus, for every gage location there is available a record showing the stresses induced by adding a foot of concrete at successive elevations. This is a continuous function which can be integrated graphically or numerically to indicate the effect of successive stages of construction, including the dead load stresses in the complete dam. Thus, critical stages in the construction when dead load stresses are a maximum which exceed the stress state in the completed dam will be indicated by study of the graph of stress versus height of dam.

The general relationships developed in using the method of integration are shown in Fig. 15. At the left is shown a drawing of the model,

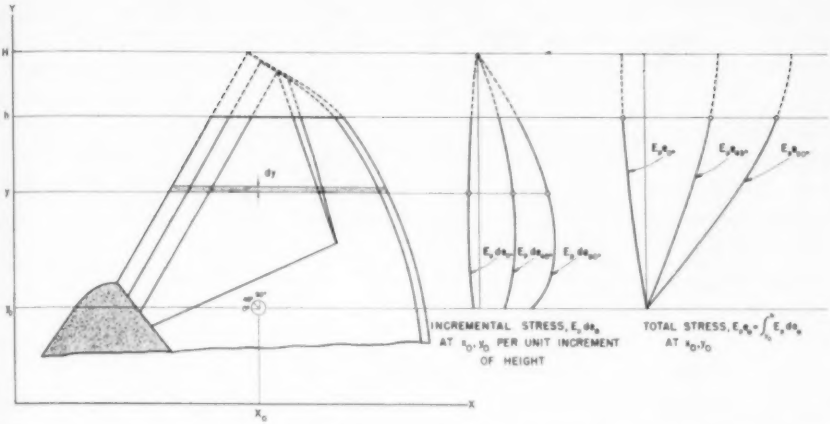
INCREMENTAL STRESS, $E_s d\epsilon_s$ AT x_0, y_0 PER UNIT INCREMENT OF HEIGHT
TOTAL STRESS, $E_s \epsilon_s = \int_{y_0}^{h} E_s d\epsilon_s$ AT x_0, y_0

Fig. 15—Notation for method of integration for determining dead load stresses

with ultimate height H, and constructed to elevation h. At elevation y_o a strain gage rosette was placed and at elevation y an increment of elevation dy shown. If the model is cut down to a horizontal surface at elevation y and that surface loaded uniformly, the incremental stress computed from strains measured on the strain gage rosette can be plotted as shown on the incremental stress plot. If tests are performed on enough elevations y, a continuous function can be drawn. If this continuous function is integrated upward, a graph of total stress can be drawn which represents, at any height, the stress in the direction of the elements of the strain gage rosettes due to constructing the dam to the height h under consideration. The values at the top of the graph represent the dead load stresses for the completed dam. These stress elements can then be combined to find the magnitude and direction of the principal stresses at that location.

Incremental loading and strains in model and prototype are related by:

$$de_{yp} = \frac{E_m}{E_p} \cdot \frac{\gamma_p}{W_{ym}} e_{ym} dy \qquad (6)$$

where W_{ym} is the intensity of loading on the model at elevation y, and γ_p is the density of the prototype.

Thus the dead load stress for the prototype completed to elevation h will be

$$\sigma_{hp} = E_m \gamma_p \int_{y_o}^{h} \frac{e_{ym}}{W_{ym}} dy \qquad (7)$$

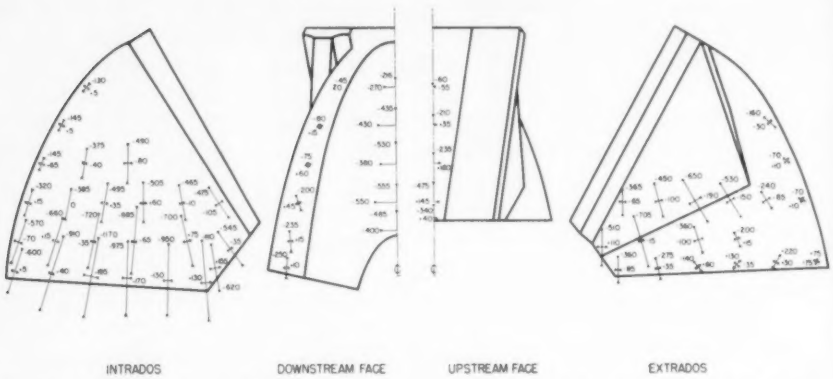

Fig. 16—Distribution of principal stresses under dead load of completed dam

Dead load stresses

Fig. 16 shows the direction and magnitude of the dead load stresses determined for the completed structure. For the intrados, the principal compressive stresses increase gradually from top to bottom, with the directions parallel to the upstream or downstream boundaries, varying gradually across the cross section. For the extrados, the discontinuity introduced by the riding buttress is evidenced by higher stresses at its lower boundary. Fairly high tensile stresses are found near the boundary of the arch and plug for both extrados and intrados, indicating that bonding these two structures at the boundary served to develop tensile forces that would not have been present in the prototype with an open construction joint. The maximum compressive stress is 1170 psi near the center of the intrados in the vertical direction. The maximum tensile stress of 220 psi is found near the downstream end of the extrados, where the effect of the inward-leaning arch is felt.

As an example of how the method of integration can be used to study the effect of a given construction program on stresses in a dam, Fig. 17 shows the vertical stresses at two elevations on the intrados and extrados of the dam. It can be seen that at both elevations shown on the intrados, the vertical compressive stresses increase continuously as the dam is constructed upward. However, a different picture is presented when considering the vertical stresses at the extrados. The stresses at Elevation 450 increase continuously with construction of the dam except for the extreme upstream location where hardly any stress is registered. However, at the base the vertical stresses are slightly compressive at the upstream locations, somewhat tensile at the downstream locations, and vary from tension to compression near the center of the base. This indicates qualitatively that the contribution of load

from the riding buttress must be causing some twisting of the arch near the base. Similar inferences can be drawn by making a corresponding study of the development of principal stresses as construction proceeds upward.

Study of Fig. 2 showing the general elements of the dam discloses that horizontal arch sections are continuous above Elevation 400 and discontinuous below that elevation. This indicates that much of the strength of this type of dam lies in the continuity of the arch and its

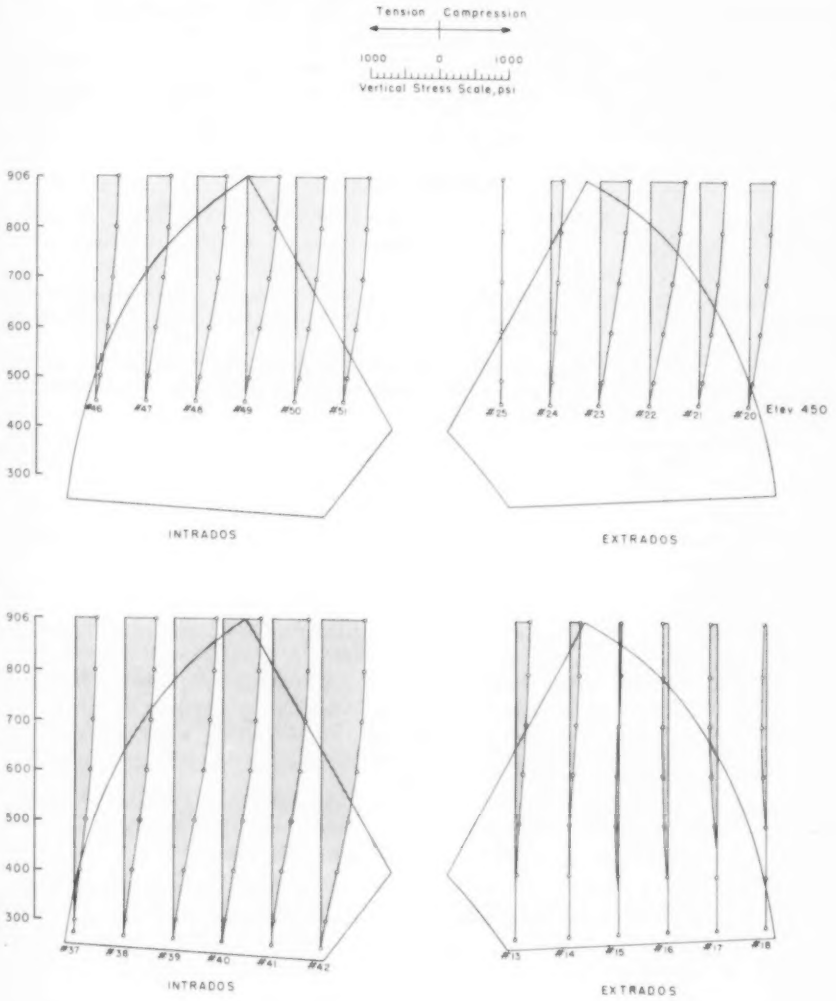

Fig. 17—Vertical dead load stresses at various stages of construction

crown. Hence in laying out the construction program for the dam, consideration should be given to developing this strength through the crown as early as possible. This might indicate the use of inclined arches for construction joints. This serves further to illustrate the usefulness of the method of integration in determining the effect of construction procedures on the final stresses in a concrete dam.

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Ultimate Strength of Square Columns Under Biaxially Eccentric Loads

By RICHARD W. FURLONG

The ultimate strength capacity of square columns under biaxially eccentric loads is investigated using Whitney's equivalent rectangular stress distribution. A description of ultimate strength behavior is presented in the form of column interaction diagrams at various skew angles of eccentricity.

■ THE PROBLEM OF DESIGNING reinforced concrete members to carry compression and biaxial bending is quite complex. Any generalized procedure for designing such members will necessarily involve some estimating of cross sections followed by rather tedious checking of the estimates until an optimum cross section is obtained. The checking procedure is especially complex for members in which eccentric loads act in planes other than planes of symmetry for reinforced concrete members. Procedures and graphs for calculation aid in the ultimate strength analysis of rectangular members under skew bending have been presented by Au,[1] and a general procedure for any shape under skew bending was formulated by Chu and Pabarcius.[2] In all accurate procedures, the labor of checking successive approximations is extensive.

To reduce the labor required to predict the behavior of columns with skew bending, some insight can be developed by studying the behavior of typical column shapes. Similar shapes will exhibit similar characteristics which, when correlated, can save much time in design and can indicate some rules for guidance with specifications governing design. The most frequently encountered column shapes are circular or square cross sections. Circular cross sections present no extra difficulty due to skew bending because circular cross sections have the same resistance to bending in all directions.

The purpose of this paper is to describe the ultimate theoretical capacity for square, tied columns to carry biaxially eccentric compression loads. Columns with equal amounts of reinforcement in each of four faces are studied, and data are presented for such columns as well as for square columns with reinforcement in opposite faces only. (The columns with reinforcement only in opposite faces indicate the trend of behavior for rectangular shapes in which there is unequal bending resistance about the principal axes.)

DESCRIPTION OF INVESTIGATION

The ultimate strength of eccentrically loaded columns is governed by the crushing resistance of concrete, assuming that reinforcing bars are ductile enough to deform at constant yield stress until concrete crushes in compression. ACI-ASCE Committee 327, Ultimate Strength Design,[3] recommends a maximum strain of 0.003 at which concrete may be depended on to resist crushing. With a maximum extreme fiber strain of 0.003 in compression, and the accepted assumption that plane sections before bending remain plane after bending, it is possible for any assumed neutral axis to describe strains at all points of a cross section. The plane of the deformed section at ultimate load is established as that plane passing through the neutral axis (a line of zero strain) and the point of maximum compressive strain as shown in Fig. 1.

Whitney's stress block

With strains at all points established, and with knowledge of stress-strain characteristics for the reinforcement used, it is possible to evaluate the resultant force and moment that created the assumed strain distribution. Steel was assumed to possess a trapezoidal stress-strain diagram in which steel stress f_s equals the product of strain, ε_s and Young's modulus E until strains exceed the yield strain of steel. In both tension and compression, for all strains beyond the yield strain, steel stress was assumed constant at the minimum yield stress specified for the type of steel considered. Concrete was assumed to have no tensile strength, and the resultant compression force on concrete was approximated using Whitney's equivalent rectangular stress distribution. Whitney's rectangular stress block is probably the most convenient aid in estimating the volume and locating the centroid for a stress "wedge" of concrete at failure conditions (a somewhat parabolic wedge to be sure). Ultimate strains greater than 0.003 in. per in. can be expected when maximum strain exists only at the one point representing the apex of a triangular area.[4] As long as calculations are based on a maximum strain of 0.003 in. per in., the assumption of a constant maximum stress throughout a triangular area is reasonable and safe because the assumed area of application of stress, therefore, is smaller than the actual stressed area to be expected at failure. In the application of Whitney's rectangular stress distribution, restrictions of Committee 327 were observed in using an average stress of 85 percent of the cylinder strength of concrete ($0.85 f'_c$) and limiting the depth of stress block a in the direction of the neutral axis to 85 percent of the distance from the extreme fiber to the neutral axis, ($a = 0.85 k_u d$). Forces resulting from an assumed strain distribution are shown in Fig. 1. Summation of forces and moments about the center of the column yield the resultant force and moment causing the assumed strain distribution.[5]

Richard W. Furlong is assistant professor of civil engineering, University of Texas, now working on his doctorate. His investigation of reinforced concrete columns on skewed bridge bents from ultimate strength standpoint led directly to this paper. Prior to assuming his present position in 1958, Professor Furlong had worked for various firms as a detailer, stress analyst, and design engineer.

If a series of neutral axes are assumed perpendicular to an axis of symmetry, a series of ultimate forces and corresponding ultimate moments can be obtained. The resultant moments will each act in the plane of the axis of symmetry. A graph of the ultimate axial forces against corresponding ultimate moments is shown in Fig. 2. Such a diagram, for loads in one plane, is called an interaction diagram.

Eccentricity

In Fig. 2, the point P_o is simply the axial load at which the concrete cross section is assumed to crush at all points simultaneously. As the load P_u is moved slightly away from the center of the cross section, the extreme fibers of concrete in the direction of eccentricity will crush before any other points of the cross section fail. Such failures are said to be compression failures. Obviously as eccentricity e' increases, P_u, must decrease, and M_u, which equals the product of P_u and e', increases. There is a particular eccentricity for any cross section and plane of bending at which the extreme fibers of concrete in compression crush at the same instant that the first (or extreme) reinforcing bar reaches its yield strength. The load and moment at such an eccentricity are called the "balanced" load and balanced moment at a balanced eccentricity. As eccentricity is increased beyond the balanced eccentricity, the yielding

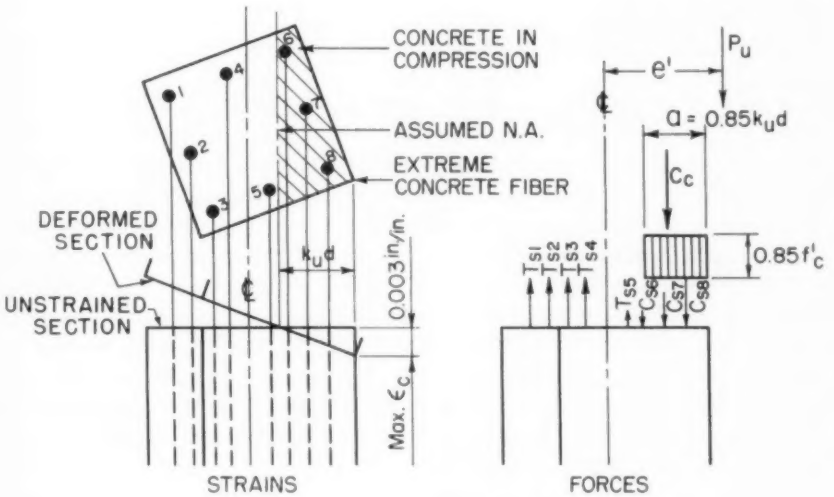

Fig. 1—Strains and resulting forces

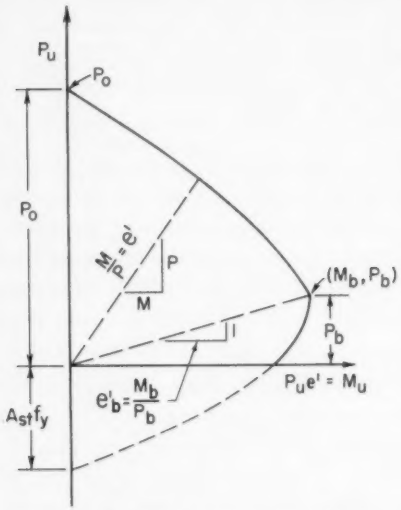

Fig. 2—Typical interaction diagram

of reinforcement precedes crushing of concrete and such failures are considered to be tension failures.

Because ultimate moments may be expressed as an axial load at some eccentricity from the center of the column, the slope with respect to the P_u axis of a line from the origin of the interaction diagram to a point of the diagram is equal to the eccentricity of load at that point of the diagram. At an infinite eccentricity, P_u is zero and the pure bending capacity of the cross section is determined. Then, as a limiting case for resultant P_u in tension, the tension capacity of reinforcing bars at their yield stress in tension establishes the strength of the cross section. The value, $A_{st}f_y$, representing ultimate capacity of reinforced concrete members in pure tension, is meaningless for columns, but it may serve as a convenient point for sketching interaction diagrams. The tension failure region of an interaction curve can be crudely approximated as a parabola passing through the point $P_u = -A_{st}f_y$ and tangent to a vertical line at P_bM_b.

Interaction diagrams

Interaction diagrams describe column capacity for bending in one particular plane. By varying the angles and locations of assumed neutral axes, it is possible to obtain data for a series of interaction diagrams at many angles between axes of symmetry of a cross section. A complete set of interaction diagrams at all angles will describe an interaction surface. For cross sections with two axes of symmetry, each quadrant of the interaction surface will be identical, so that one quadrant is sufficient to describe the surface. A possible interaction surface is shown in Fig. 3. The interpretation and plotting of three-dimensional figures is difficult; a more obvious description of the surface can be constructed as a series of intersections of the surface with planes of constant load P_u. As suggested in Fig. 3, moment capacity in directions of principal axes may be used as abscissas and ordinates, and a series of curves or "contours" to the surface will appear for constant values of P_u when the surface is seen from the P axis.

Columns in which bending capacity is the same about each principal axis will have identical interaction diagrams for bending in the plane

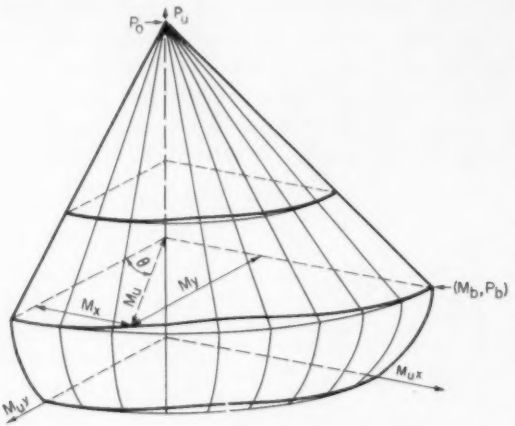

Fig. 3—Typical interaction surface

of each axis, and an interaction surface should approach a surface of revolution generated as the interaction diagram rotates about the P axis as suggested by the grid lines of Fig. 3. Contours to a surface of revolution would appear as circular arcs.

In evaluating points for an interaction diagram a cross section 20 in. sq was used. Results were generalized by dividing ultimate loads by $f_c't^2$ and ultimate moments by $f_c't^3$ for plotting. As is customary, f_c' represents cylinder strength of concrete, and t is the over-all thickness of the column cross section. Four bars were assumed in each face of the columns as shown in Fig. 4. Using a concrete cover ratio, $gt =$ 0.75t, cross sections with bars in all four faces and cross sections with bars in only two opposite faces were studied allowing variations of f_c', f_y, and steel percentage. Cylinder strengths of 3000 psi and 5000 psi were used, and yield strengths of 40,000 psi and 50,000 psi were used. In all cases steel percentages of 1 percent and 4 percent were investigated.

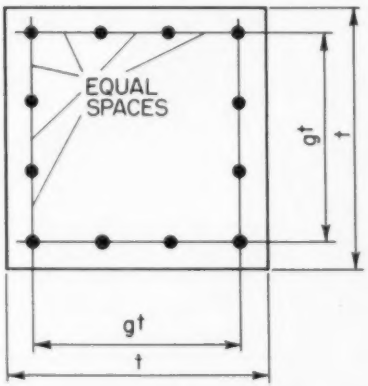

Fig. 4—Cross section used for calculation

Data for interaction surfaces was accumulated for the many variables by first assuming a series of neutral axis locations at some constant angle with the x axis. Resulting stressed areas of concrete were evaluated and multiplied by the x and y coordinates for the centroid

of each area, yielding unit force and moment (M_x and M_y) components due to concrete. Steel stresses, likewise, were tabulated for each bar as determined by its distance from the neutral axis and the slope of the deformed section. The steel stresses were then multiplied by the x and y coordinates yielding unit force and moment components due to steel. Then 0.85 f_c' (f_c' equal to 3000 or 5000 psi) was multiplied by the unit concrete components, and actual steel areas (0.01 A_g or 0.04 A_g) were multiplied by unit steel components for final addition (algebraically) to yield final force P_u and final moments M_x and M_y. Obviously, even values of $P_u/f_c't^2$ occurred only by coincidence, so that M_x and M_y versus P_u curves were constructed for the double purpose of indicating arithmetic errors and obtaining M_x and M_y values at even values of $P_u/f_c't^2$ for plotting interaction surfaces.

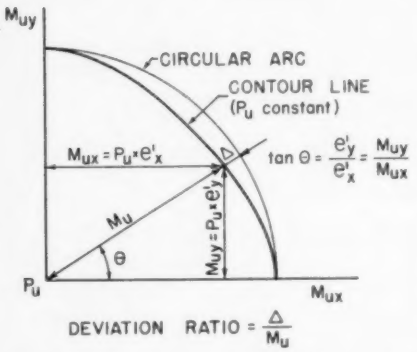

Fig. 5—Contour line at constant axial load

Although it was a questionable value in precision, the effect of concrete displaced by reinforcement was included in all cases by reducing the effective stress on compression steel by 0.85 f_c'.

Four bars were considered in each face assuming that data for such distribution of reinforcement would be consistent with corresponding data for cross sections including many more bars per face. Pannell states that columns with three or more bars per face show consistent properties under biaxial bending, but that columns with only two bars per face (a total of 4 bars in the cross section) show considerable variations from columns containing more bars.[6]

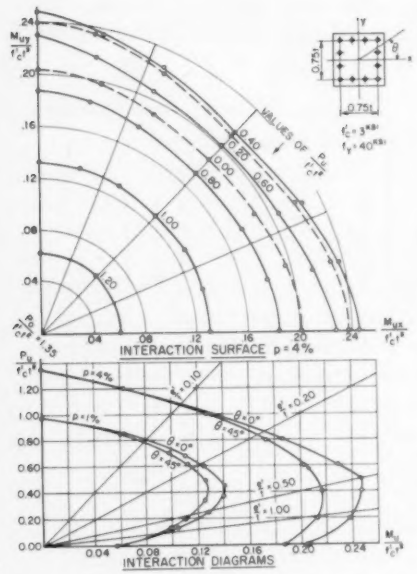

Fig. 6 — Interaction surface and projected interaction diagrams

Interaction surfaces for columns containing 4 percent reinforcement are shown in Fig. 6 through 15. Interaction diagrams at 0 deg and 45 deg from the major axis of each

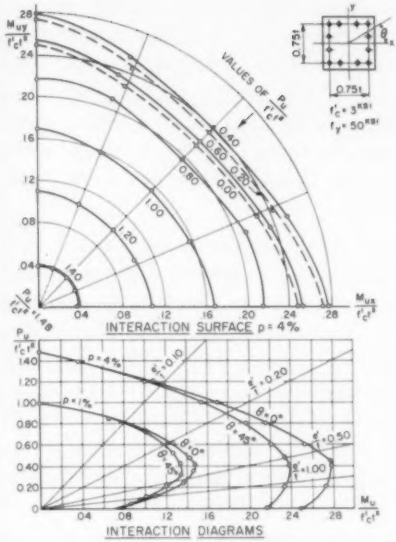

Fig. 7 — Interaction surface and projected interaction diagrams

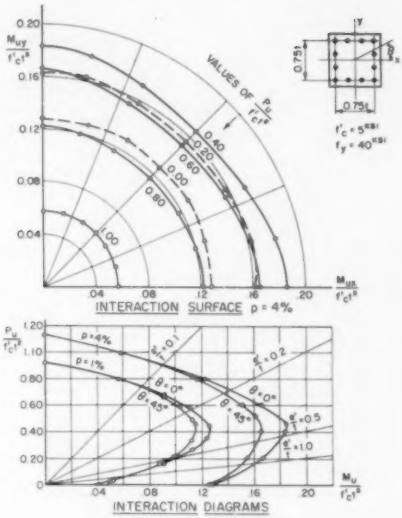

Fig. 8 — Interaction surface and projected interaction diagrams

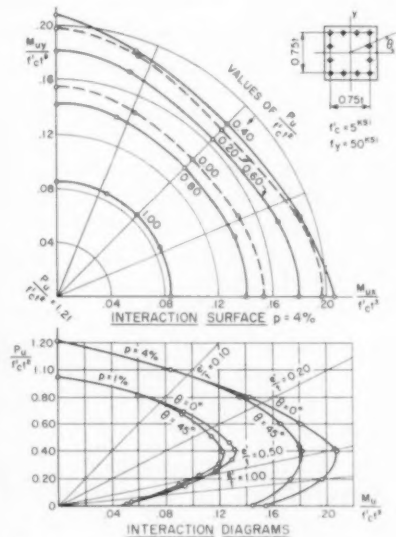

Fig. 9 — Interaction surface and projected interaction diagrams

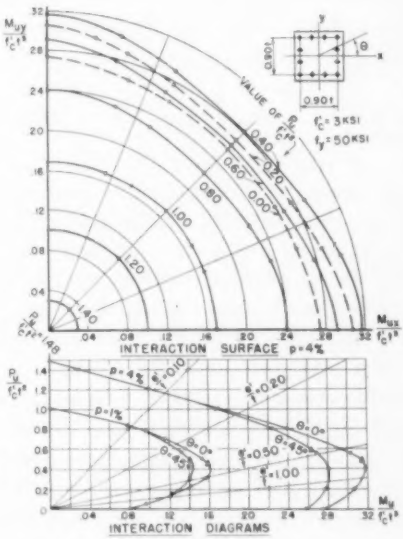

Fig. 10—Interaction surface and projected interaction diagrams

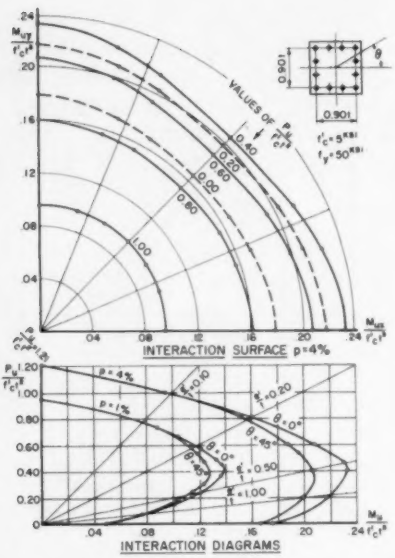

Fig. 11—Interaction surface and projected interaction diagrams

section, and with 1 and 4 percent reinforcement, are shown with each column containing steel in four faces. Interaction diagrams at 0, 30, 60, and 90 deg at 4 percent reinforcement are shown with columns containing steel in opposite faces only.

Interaction surfaces for columns containing 1 percent reinforcement were much the same as interaction surfaces for columns containing 4 percent reinforcement except that changes in moment capacity at various angles were smaller, as may be seen by comparing interaction diagrams at 0 and 45 deg. Therefore, the interaction surfaces for 1 percent reinforcement are not included in the data presented.

The influence of concrete cover ratio g on variations in the general shape of interaction surfaces was investigated also. Assuming that steel with 50,000 psi yield strength should involve greater variations in surfaces than steel with 40,000 psi yield strength, cross sections with $gt = 0.90t$ were studied and interaction surfaces for these columns are shown in Fig. 10 and 11.

DISCUSSION

At any particular load P_u, the distance from the P_u axis to a point on the interaction surface represents the magnitude of a resultant ultimate moment M_u at the particular load and angle of eccentricity θ investigated. Fig. 5 shows a fairly typical contour line. If bending resistance at the particular load P_u were constant for all angles of eccentricity, the contour line would be a circular arc. Columns with equal amounts of steel in each face have equal bending resistance about each principal axis, and contour lines of Fig 6 through 11 for such columns do tend to approach circular arcs. Comparative amounts of deviation from circular arcs may be expressed as a deviation ratio $\Delta/(M_u + \Delta)$.

For columns with reinforcement in four faces, deviation ratios appear to be highest in regions of maximum moments and at skew angles of 45 deg. The deviation ratio decreases for skew angles different from 45 deg and decreases also for smaller moments. Since maximum moments usually occur at or near the balanced load condition, it could be said

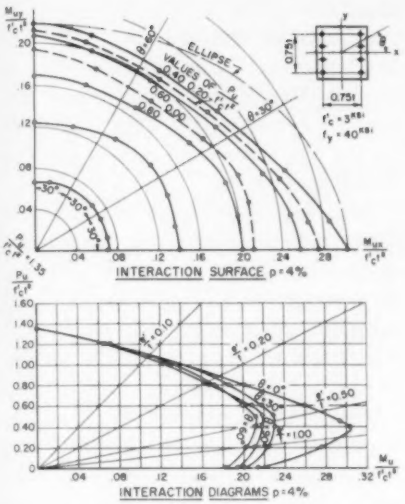

Fig. 12—Interaction surface and projected interaction diagrams

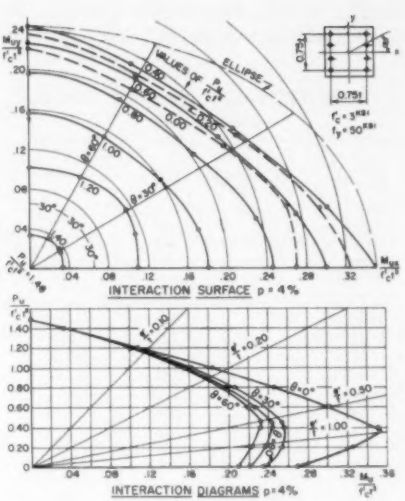

Fig. 13—Interaction surface and projected interaction diagrams

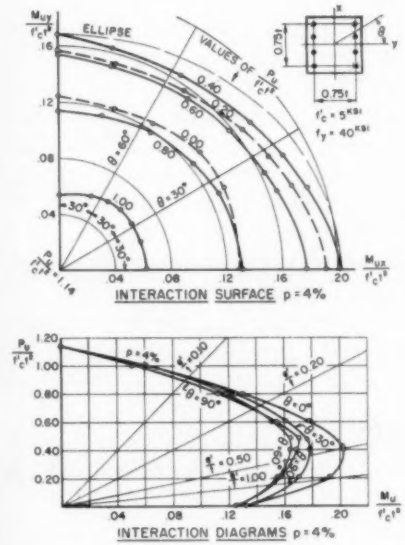

Fig. 14—Interaction surface and projected interaction diagrams

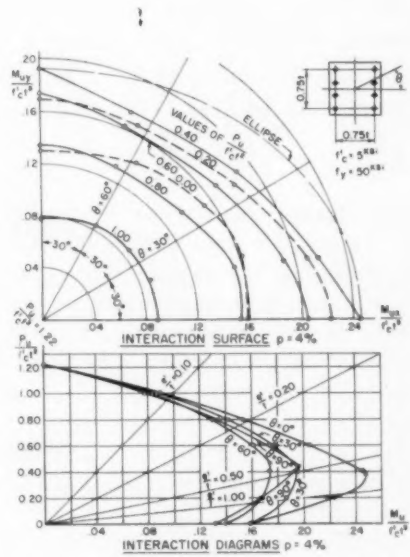

Fig. 15—Interaction surface and projected interaction diagrams

TABLE 1 — MAXIMUM DEVIATION RATIOS AT $P_u/f_c't^2 = 0.40$

REINFORCEMENT IN FOUR FACES
Circular arcs

p, A_{st}/t^2	g	f_c', ksi	f_y, ksi	Maximum deviation ratio
0.04	0.75	3	40	0.12
		3	50	0.15
		5	40	0.10
		5	50	0.12
0.01	0.75	3	40	0.09
		3	50	0.09
		5	40	0.05
		5	50	0.07
0.04	0.90	3	40	0.11
		3	50	0.12
		5	40	0.09
		5	50	0.12
0.01	0.90	3	40	0.09
		3	50	0.10
		5	40	0.09
		5	50	0.09

REINFORCEMENT IN TWO FACES
Elliptical arcs

p, A_{st}/t^2	g	f_c', ksi	f_y, ksi	Maximum deviation ratio
0.04	0.75	3	40	0.15
		3	50	0.17
		5	40	0.09
		5	50	0.15
0.01	0.75	3	40	0.11
		5	40	0.09

that deviation ratios decrease as the absolute value of $P_u - P_b$ and the absolute value of $(\theta - 45)$ increase. Virtually no deviation exists for eccentricities less than one-tenth the total thickness of the cross section.

Interaction surfaces for columns with reinforcement in only two (opposite) faces, are shown in Fig. 12 through 15. In regions of highest moment resistance, i.e., at $P_u/f_c't^2$ equal to 0.40, contour lines are compared with elliptical arcs shown with each interaction surface. Again, maximum deviation ratios from the elliptical arcs appear to take place in regions of maximum moments and at skew angles near 45 deg. As was the case with columns containing equal amounts of steel in four faces, deviation ratios decrease as the absolute value of $P_u - P_b$ increases and as the absolute value of $\theta - 45$ increases.

Maximum deviation ratios at $P_u/f_c't^2$ equal to 0.40 are tabulated in Table 1. As might be expected, maximum deviation ratios vary more with percentage of reinforcement than with any other variable considered. Magnitudes of deviation ratios vary from 0.05 to 0.11 for $p = 0.01$, and from 0.09 to 0.17 for $p = 0.04$.

Probably due to the limited accuracy maintained in calculation and plotting of interaction surfaces, no accurate variation in deviation ratios may be derived from Table 1. However, considering the relative accuracy of the theory employed for analysis, greater precision in computation does not appear justified.

CONCLUSIONS

Based on the data accumulated, the following general conclusions appear justified:

1. The capacity for square, reinforced columns to resist eccentric compression loads is a maximum when loads cause bending about the major axis of the cross section.

2. The capacity of a column to resist moments about any axis is virtually constant for eccentricities less than one-tenth of the thickness of the column.

3. The capacity for square columns with equal amounts of steel in each of four faces to resist eccentric compression loads is smallest when loads cause bending about an axis at 45 deg from a major axis.

4. Design ultimate moment capacity for square columns at a particular load P_u should in no case exceed the limits of an ellipse whose axes are ultimate moment capacities when the load P_u acts along the principal axes of the column cross section. In the form of an equation, $(m_x/M_x)^2 + (m_y/M_y)^2$ must not exceed unity.

Where:

m_x = moment component in direction of major axis
m_y = moment component in direction of minor axis
M_x = moment capacity when P_u acts along the major axis
M_y = moment capacity when P_u acts along the minor axis

Until laboratory test data are accumulated, and more precise evaluation of biaxially loaded column capacity may be established, it is recommended that columns designed for biaxial bending be checked by the equation above. In addition, a reduction in the limiting amount of $(m_x/M_x)^2 + (m_y/M_y)^2$ from 1.00 to 0.85 or 0.90 may be appropriate if the ratio $|P_u - P_b|/P_b$ is less than 1.00 for the cross section considered.

5. The highest deviation ratios, which exist for columns reinforced in only two faces, suggest that even greater reductions in P_u and M_u from an elliptical arc may be necessary for rectangular shapes under skew bending.

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American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Mich.

Discussion of this paper should reach ACI headquarters in triplicate by June 1, 1961, for publication in the Part 2 September 1961 JOURNAL.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Building the towers of the Tamar Suspension Bridge

The Engineer (London), V. 210, No. 5454, Aug. 5, 1960, p. 242

Reviewed by ARON L. MIRSKY

Pictorial progress report of the construction of the reinforced concrete towers of the 1100 ft span suspension bridge over the Tamar, in England. Main towers, 240 ft high, comprise "broken squares" 2 ft thick with rounded corners, tapering from 14 ft at the base to 9 ft at the top.

A brief description of the climbing cranes on these towers is given in *Engineering* (London), V. 190, No. 4926, Sept. 16, 1960, p. 380.

Concreting techniques in Russia. 1—Two-level bridge in Moscow. Complexities of skew span structure

V. G. ANDREIEV, G. I. ZINGORENKO, and M. M. RUDOMASIN, *Contractors Record* (London), V. 70, No. 36, 1959, pp. 18, 21, 23, 25

ROAD ABSTRACTS V. 27, No. 6, June 1960

This bridge over the Moscow River, consisting partly of reinforced concrete and partly of prestressed concrete, carries an underground railway on the lower level and a carriageway nearly 70 ft wide and two footways, each over 7 ft wide, on the upper level. The total length of the crossing from the beginning of the embankment on one bank of the river to the end of the cutting on the opposite bank is over 1¼ miles, the three spans across the river being about 150, 350, and 150 ft long, respectively. The design and construction of the bridge are described; over 60 percent of the concrete used was precast.

Bridge construction (Brueckenbau)

FRITZ LEONHARDT, *VDI Zeitschrift* (Düsseldorf), V. 102, No. 18, June 21, 1960, pp. 735-742

Reviewed by ARON L. MIRSKY

Biennial survey* of the state of the art, as exemplified by developments in design and construction of bridges of various materials and types.

*The preceding survey was published in *VDI Zeitschrift*, V. 100, No. 18, June 21, 1958, pp. 784-788 ("Current Reviews," ACI JOURNAL, V. 30, No. 7, Jan. 1959 (*Proceedings* V. 55), p. 818.

Construction

Wire mesh reinforcing heats slab, too

Building Construction, V. 30, No. 5, Nov. 1960, pp. 26-27

A 6500 sq ft electrified concrete floor slab uses 6 in. x 6 in. 4-, 6-, and 8-gage standard wire mesh to reinforce as well as heat a 6 in. thick concrete slab-on-grade, placed over a vapor barrier and 6 in. gravel fill. Construction of electrified slab is pictured.

Dungeness Lighthouse

The Engineer (London), V. 210, No. 5453, July 29, 1960, pp. 206-207

Reviewed by ARON L. MIRSKY

130 ft high tower is structurally distinguished by its construction: the shell of the tower is formed of 21 precast concrete drums 5 ft high, 12 ft in out-

A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 9, Mar. 1961, *Proceedings* V. 57. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are *not* available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

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side diameter, and 6 in. in wall thickness, placed by crane and tensioned by steel wires run through vertical ducts from top to bottom. After tensioning the wires were grouted and the effectiveness of the grouting checked by radiography with a Cobalt-60 bomb. Tower has a distinctive banded appearance, obtained by using black and white concrete mixes designed for the job. The base, in white concrete, has the form of a spiral ramp and contains the machine room.

Construction Techniques

Concrete floors for houses

N.Z. Concrete Construction (Wellington), V. 4, No. 8, Sept. 12, 1960, pp. 120-134

A series of brief articles explains the fundamental principles involved in the design and construction of concrete floors for residential work. Typical drawings and diagrams are included. Subjects covered include site preparation, foundation walls, granular base, control of moisture, concrete slab, provision for services, attachment of superstructure, and floor finishes and surface coverings. The subjects of heating and insulation, underground drainage systems, and specifications of materials are not covered in these articles although it is mentioned that appendices covering this material are available.

Using shell piles to reach a firm foundation

Municipal Journal (London), V. 67, No. 3443, 1959, pp. 450-451. *Building Science Abstracts*, V. 32, No. 12, Dec. 1959, p. 365

HIGHWAY RESEARCH ABSTRACTS V. 30, No. 8, Sept. 1960

The pile described comprises a precast reinforced concrete tube in sections, threaded on to a steel mandrel, at the bottom of which is a solid concrete shoe. The whole unit is driven by drop-hammer to a predetermined and calculated set, the steel mandrel then being removed and a reinforcement cage inserted into the tube; concrete is then placed to form an unstressed and solid core. Any number of sections may be added to obtain a required length, and the sections are interconnected by metal bands around recesses in their ends. The system is capable of definite mathematical treatment by the Hiley formula, and the interior of the pile can be inspected after driving. Piles can be driven 50 ft or more below the surface and through poor subsoil; vibration and noise are less with this system than with any other driven pile. Some applications of the system are described and illustrated.

Vacuum concrete method on the construction site (Das Vacuum-Concrete-Verfahren auf der Baustelle)

E. SEHORSCH, *Beton-Herstellung und Verwendung* (Düsseldorf), V. 10, No. 8, Aug. 1960, pp. 363-370

Reviewed by FERDINAND S. ROSTASY

Describes the mechanics involved in the vacuum concrete method. Through suction exerted by the vacuum form on the fresh concrete, tensile forces are set up in its capillaries. On removal of the vacuum formwork the capillary forces are preserved in the concrete. The vacuum concrete method yields a series of advantages: fast re-use of the expensive forms, increase of strength, greater abrasion resistance and imperviousness; and shrinkage is reduced. Equipment necessary for vacuum concreting on the site is discussed. Several examples are described in detail, such as sewage tanks and precast elements. Hereby excess water is removed and the aggregate structure densified.

Design

Reinforced concrete reservoirs and tanks

W. S. GRAY, Concrete Publications Limited, London, 4th Edition, 1960, 190 pp., $2.80

The 4th edition of this book has been extensively revised to include up-to-date methods of the design and construction of most types of open and covered reservoirs, tanks on the ground and below ground, swimming pools,

and such industrial structures as gasholder tanks, tar tanks, and the like.

The book includes not only broad principles of design but also discusses various methods of construction and building details. The appendix includes notes on the British Standard Code of Practice for the Design and Construction of Reinforced and Prestressed Concrete Structures for the Storage of Water and Their Aqueous Liquids (1960).

Stepwise constant stress distribution (in Swedish)

A. AAS-JAKOBSEN, *Nordisk Betong* (Stockholm), V. 4, No. 3, 1960, pp. 209-230
Reviewed by MARGARET CORBIN

Slender structural members subjected to eccentric compression are designed for the given cross-sectional forces and additional moments by an equation where σ_E is the stress corresponding to the characteristic value of the structure for a material having a modulus of elasticity E_c. Columns subjected to central loads are designed for the additional moments and the allowable stresses increased by 15 percent when $l/i > 40$. Walls, shells, and similar structures fixed in two directions x and y, which are acted upon by the respective components N_x and N_y of the two-axial normal force, are submitted to an additional moment in each of these two directions given by an equation.

Plastic theory versus elastic theory (in Hebrew)

A. ZASLAVSKY, *Handasa We-adrikhalut* (Journal of Association of Engineers and Architects in Israel), V. 18, No. 9, Oct. 1960, 6 pp.
AUTHOR'S SUMMARY

Reviews the basic difference between the two methods and recommends the (exclusive) incorporation of the plastic method into the proposed Israeli Code of Practice. Reinforced as well as steel structures are dealt with. Both statically determinate and indeterminate structures are analyzed. In the latter, the redistribution of moments is of special importance, but in reinforced concrete this redistribution is limited by the rotational capacity of the plastic hinges.

Codes of practice of different countries are reviewed, as well as their terminology.

Numerical examples are worked out by both methods to show the wide variation of the safety factor (against plastic failure) of different structures designed by the elastic theory.

The plastic theory, satisfying the strength conditions only, has its limitations and does not, therefore, supersede entirely the elastic theory which must be used in the analysis of the working stage of the structure.

Analysis of static schemes for the use of prefabricated ferro-concrete pit prop structures (in Russian)

L. Y. KHMEL'NITSKII, P. S. IVANOV, and V. A. BONDAREV, Investigations on Mine constructions, Ugletekhizdat, Moscow, 1958, pp. 83-96; *Referativnyi Zhurnal Mekhanika* (Moscow), No. 6, 1959, Rev. 6894
APPLIED MECHANICS REVIEWS
V. 13, No. 11, Nov. 1960

This is an investigation on the selection of rational static schemes of mine-prop structures to withstand rock pressures while taking into account the elastic resistance of the rock stratum. Graphs are drawn of the forces in question for pit props of trapezoidal, polygonal, and circular outlines. A comparison of the different forms showed that the most rational was the assembly of polygonal outline and of braced construction for the pit prop, with hinges at the joints which enable the prop to "adapt itself" to the action of the rock pressure.

Simply supported corner plate

MARIO G. SALVADORI and H. G. REGGINI, *Proceedings*, ASCE, V. 86, ST11, Nov. 1960, pp. 141-154
AUTHORS' SUMMARY

A corner plate, simply supported on two parallel boundaries, each composed of two semi-infinite straight lines meeting at right angles, is uniformly loaded. Bending and twisting moments and shears and boundary reactions in the plate are determined by finite-

difference integration of the corresponding boundary value problem. The penetration into the side strips of the disturbance represented by the corner is shown to be practically negligible at a distance from the inner corner of the order of the width of the side strips.

Certain aproximate analyses of the nonlinear behavior of plates and shallow shells

WILLIAM A. NASH and JAMES R. MODEER, *Proceedings*, Symposium on the Theory of Thin Elastic Shells, Delft, Holland, 1959 (Reprinted as *Technical Paper* No. 193, Florida Engineering and Industrial Experiment Station, Gainesville, 1960, 24 pp.)

AUTHORS' SUMMARY

Approximate equations governing the nonlinear behavior of (a) flat plates vibrating with finite amplitudes, (b) the equilibrium of shallow spherical shells loaded by uniform normal pressure on the concave face, and (c) the buckling of shallow spherical shells loaded by uniform normal pressure on the convex face are derived. The characteristic feature of all these sets of equations is that they are uncoupled; i.e., it is possible in each case to solve a single equation for the deflection normal to the middle surface of the plate or shell. This is in contrast to existing and somewhat more precise investigations which result in coupled sets of equations.

The present work is based on the technique of neglect of the second invariant of the middle surface strains in the expression for the total potential energy of the system. Application of a variational procedure yields the approximate, uncoupled equations. In all three cases the results of this approximate analysis are shown to be in excellent agreement with more precise analyses which involve much more computational effort than is required by the approximate methods. The present study may be regarded as an extension of work carried out by Berger with regard to static deflections of flat plates. Lastly, an improvement is offered upon Berger's technique by retention of the linear terms appearing in the second invariant.

Plates supported on two adjacent edges subject to uniformly distributed loads (Die an zwei benachbarten Rändern gelagerte Platte unter Gleichlast)

K. STIGLAT and H. WIPPEL, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 4, Apr. 1960, pp. 89-93

Reviewed by RUDOLPH SZILARD

Moments reactions, and deflections of plates supported on two adjacent edges (one of them is freely supported, the other one restrained) have been computed using differential equations of the theory of elasticity. Results from span ratios 0.4, 0.75, 1.0, and 1.33 are tabulated.

Design of the shell roof of airplane hangar No. 3 at the Frankfurt/AM airport (Zum Entwurf des Schalendaches der Flugzeughalle III auf dem Flughafen Frankfurt/Main)

GEORGE KIRCHNER, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 4, Apr. 1960, pp. 73-81

Reviewed by RUDOLPH SZILARD

The increasing jet airplane traffic required new hangars at the airport in Frankfurt/AM to accomodate large airplanes. Extensive preliminary design has indicated the advantages of cantilever type hangars. Based on U.S. designs, eight different schemes have been investigated and compared. The final design has utilized the structural characteristics of concrete carrying mainly compression and the structural characteristics of the steel carrying tension. To increase the elastic stability and reduce the dead load, double curved cylindrical shell type structures have been used in the cantilever parts of the hangar. The over-all stability against overturning has been checked for unsymmetrical wind and snow load including no counteracting dead load on the other side. The center line of the shell structures follow the compression line. The effect of a deviation of ± 10 cm has been checked analytically. The typical shell section is a "butterfly" section with continuous opening at the crown to accommodate roof windows.

DISCUSSION, PROCEEDINGS V. 57

Discussion of papers published in the July through September, 1960, JOURNALS appears in the concluding pages of this March issue, as the Institute continues its quarterly publication of discussion. Papers published October through December, 1960, will be discussed in the June 1961 issue; discussion of papers published January through March, 1961, will be published in Part 2, September 1961 ACI JOURNAL. Discussion of April through June papers will appear in Part 2, December 1961 along with index and errata for V. 57.

Discussion of a paper by Hubert Rüsch:

Researches Toward a General Flexural Theory for Structural Concrete*

By IQBAL ALI, EIVIND HOGNESTAD, LADISLAV B. KRIZ, R. G. SMITH, and AUTHOR

By IQBAL ALI†

The writer some time ago presented a rather limited analytical study‡ wherein the ultimate flexural strength of rectangular reinforced concrete sections was intepreted in terms of the stress-strain relationship of concrete under axial compression. An idealized curve was used for the analysis. Ultimate strength was identified with stress distribution resulting in the maximum internal moment. It was demonstrated that, even for the same concrete, the maximum compressive strain is by no means a constant, and is influenced to a considerable extent by the location of the neutral axis. Conclusions, identical to those of the author, were reached regarding the apparent discrepancies between the values of the ultimate concrete strain in flexural compression, as observed by different workers.

Professor Rüsch is to be complimented on his lucid exposition of a very significant contribution towards a better understanding of the behavior of structural concrete in flexure at ultimate loads. Of special importance are the investigations on the influence of the strain rate on the stress-strain response of concrete and the studies on the behavior and strength of concrete under sustained loads. The design curves presented are ingenious and practical.

The derivation of the stress-strain relationship of concrete under sustained load, from observations on specimens under eccentric compression and the suggested approach for determination of the ultimate strength of structural members on this basis, may however need some further elucidation.

*ACI Journal, V. 32, No. 1, July 1960 (*Proceedings* V. 57), p. 1. Disc. 57-1 is a part of copyrighted Journal of the American Concrete Institute, V. 32, No. 9, Mar. 1961 (*Proceedings* V. 57).

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‡Ali, I., "The Ultimate Flexural Strength of Reinforced Concrete—A New Approach," *Indian Concrete Journal* (Bombay), V. 33, No. 3, Mar. 1959, pp. 83-87, 104.

Ultimate load design involves the concept of a load, which if imposed, would result in failure of the structural member. The available margin of safety is then defined in relation to the anticipated service loads and this ultimate load. The ultimate load may be considered as a purely fictitious load, larger in magnitude by a specified margin than the normally expected service load, but essentially similar in nature and distribution, irrespective of the possibility of its actual occurrence. Alternatively it could be considered as a possible "catastrophic" load to which a structure might be actually exposed and suffer failure. Such loads can be entirely different in nature from the normal service loads. Impulsive or shock loads due to seismic or other sources, vibration stresses due to wind or waves, and stresses due to settlement, are a few of the possible causes that can result in structural failure. The assumption of a sustained over load as suggested by the author may therefore not be justifiable, on grounds of realism, in all cases. It may, under certain circumstances, not even correspond to the worst situation.

The construction of the stress-strain curve for concrete under sustained load, from tests on specimens under eccentric compression, is very ingenious. The maximum eccentricity used does not, however, exceed 1/10 of the lateral dimension of the test specimen, which corresponds to a location of the neutral axis almost at the edge of the section. It is therefore felt that flexural tests on reinforced concrete beams would have provided useful additional data for developing the desired stress-strain relationship.

Design curves shown in Fig. 23 include constant strain as well as sustained load conditions. It is presumed that in the former case the descending portion of the stress-strain curve has been included for analysis, and the ultimate moments derived by maximizing the resisting moment of the concrete compressive stresses about the tensile reinforcement. It may be pointed out in this case, that when compressive reinforcement is used, the total resisting moment need not necessarily reach a maximum value, simultaneously with the moment of the concrete stresses considered by themselves, unless the compression steel yields before the ultimate moment is reached, and has a practically constant yield stress. In other cases therefore, the design curves will give a somewhat over-designed section, when compressive reinforcement is involved.

The emphasis laid by the author on the necessity of an intensive study of the stress-strain response of concrete under various conditions of loading and for different types of concrete is highly significant. The writer is convinced that in this direction lie the major clues to a fuller understanding of the strength of structural concrete, not only in flexure and compression, but perhaps also in the more elusive modes of shear and torsion.

The writer would like to take this opportunity to remark that the problem of determining realistic ultimate load conditions for use in design is not receiving its due share of study and thought. The use of almost arbitrarily chosen values of load factors in conjunction with highly refined techniques of ultimate strength analysis is, to say the least, incompatible.

By EIVIND HOGNESTAD*

The paper by Professor Rüsch presents a broad outline of the most rigorous flexural theory for structural concrete known at the present time. In some respects, the paper represents a milestone in evolution of prior work carried out in many parts of the world and leading to the first practically useful forms of ultimate strength design. In its important aspects, however, the paper is a pathfinder for researches of the future toward improvement and generalization of present design methods.

Several new contributions have been made to general flexural theory after adoption of the 1956 ACI Building Code, ACI 318-56. Whenever such new information has become available, its implications in terms of the ultimate strength design criteria of the 1956 Code appendix have been evaluated,[10-12] and repeated confirmations of the 1956 criteria have resulted. It is the purpose of this discussion again to re-evaluate the 1956 criteria and current American ultimate strength design practice in the light of Rüsch's far-reaching new concepts and test data.

RECTANGULAR STRESS DISTRIBUTION

The 1956 ACI Code specifies that: "The diagram of compressive concrete stress distribution may be assumed a rectangle, trapezoid, parabola, or any other shape which results in ultimate strength in reasonable agreement with comprehensive tests." However, the specific design equations given in the 1956 Code imply the use of a rectangular distribution, and this distribution has become widely used in American everyday design practice.[13] It has commonly been assumed that the stress intensity of the rectangular stress block equals the maximum stress of the actual curved stress distribution to be approximated; a stress not to exceed 0.85 f_c' is given by the 1956 Code. This limitation in the use of the rectangle is not strictly necessary.

The actual curved stress distribution sketched in Fig. A can be characterized by a stress coefficient k_3 giving a maximum stress of $k_3 f_c'$, an integration coefficient k_1 giving an internal compressive force of

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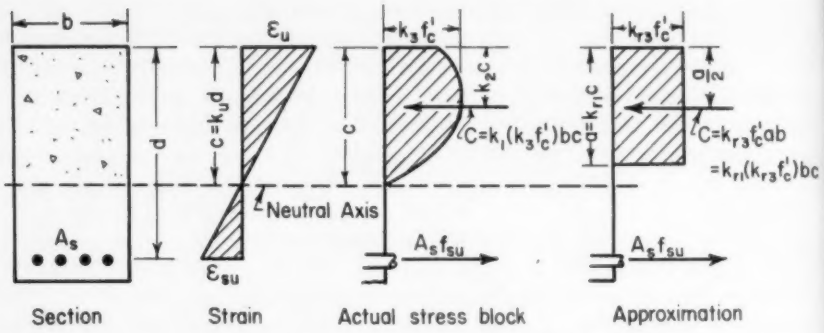

Fig. A—Nature of rectangular stress block

$k_1(k_3 f_c')\,bc$, and a centroid coefficient k_2 giving a centroid depth of $k_2 c$ for the internal compressive force. For rectangular cross sections, ultimate strength calculation by a generalized rectangular stress distribution can be made identical to those resulting from an actual curved distribution, in spite of the fact that the geometry of the rectangle always gives a centroid depth, $a/2$, equal to one-half of the stress block depth, $a = k_{r1}c$. To obtain such identical calculation, the centroid depth and the total internal force must both be identical for the two stress distributions, so that

$$\frac{a}{2} = \tfrac{1}{2}\, k_{r1}c = k_2 c \quad \text{and} \quad k_1 k_3 f_c' bc = k_{r1} k_{r3} f_c' bc$$

which gives $k_{r1} = 2k_2$ and $k_{r3} = k_3(k_1/2k_2)$. The same linear strain distribution is used for both the actual and the rectangular stress distribution.

When a rectangular stress distribution so determined is applied to members with nonrectangular compression zones, mathematical identity with solutions based on an actual curved distribution will no longer exist. This is particularly so since, as shown in the Rüsch paper, the actual stress distribution corresponding to maximum ultimate moment varies with cross section shape and depth of the neutral axis. Comparisons of calculations and new test data for nonrectangular members have shown, however, that in such cases the rectangular distribution still leads to a good approximation in calculations of ultimate strength.[14]

SUSTAINED LOAD

Rüsch's Fig. 24 "probable ultimate strength design chart for sustained load on rectangular members" is reproduced in Fig. B. A corresponding rectangular stress distribution was derived as follows. For the neutral axis at the centroid of the tension steel, $k_u = 1.0$, Rüsch's chart gives $j_u = 0.62$. The rectangular distribution gives $j_u = 1 - \tfrac{1}{2}k_{r1}$, which leads

to $k_{r1} = 0.76$. Then $m_u = 0.358 = k_{r1}db\,(k_{r3}f_c')\,j_u d/bd^2f_c'$, which gives $k_{r3} = 0.76$. Finally, for $k_u = 0.4$, Rüsch's chart gives $\varepsilon_{su} = 0.010$. Hence, $\varepsilon_u/0.4 = \varepsilon_{su}/(1 - 0.4)$, which yields $\varepsilon_u = 0.0067$.

The values $k_{r1} = k_{r3} = 0.76$ and $\varepsilon_u = 0.0067$ so determined were then used to calculate all points shown in Fig. B. It is seen that an extremely close agreement with the Rüsch chart results, except when k_u is greater than about 1.1, that is, when the neutral axis falls outside the cross section. It will be shown later that, because a minimum eccentricity is specified, such positions of the neutral axis have little practical significance in American ultimate strength design.

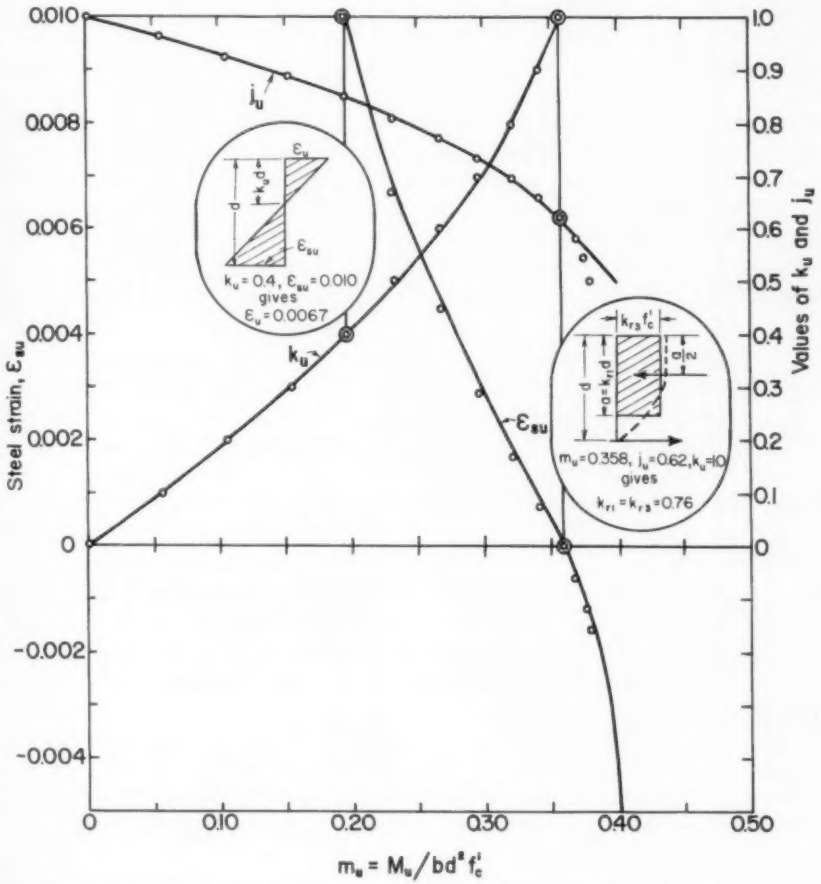

Fig. B—Rectangular stress block approximation of Rüsch's Fig. 24, sustained load

CONSTANT RATE OF STRAIN LOADING

The tests of reinforced concrete beams and columns, on which the ACI 318-56 design criteria are based, have usually been carried out in intervals of time from 1 hr to a few hours. In Fig. C, therefore, comparison is made with the curves of Rüsch's Fig. 23 corresponding to total testing times of 1 hr and 1 day. The characteristics of the rectangular stress distribution used to compute the coordinates for the points

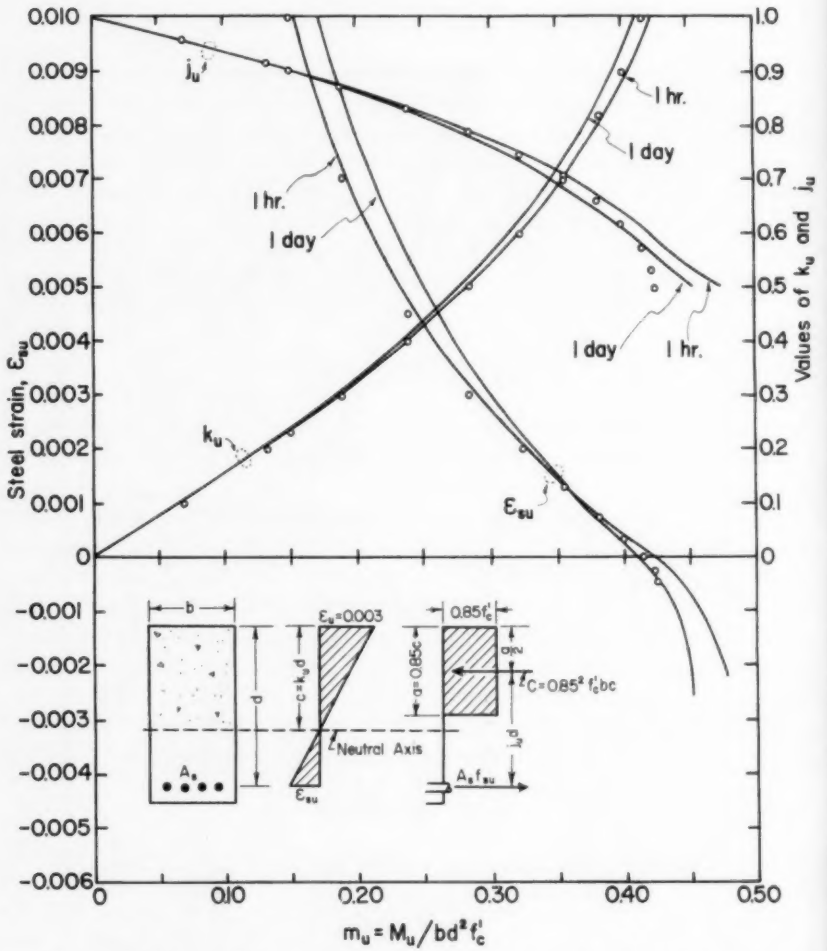

Fig. C—Comparison of ACI 318-56 rectangular stress block to Rüsch's Fig. 23

plotted in Fig. C were not derived from Rüsch's curves as they were in Fig. B. The values $k_{r1} = k_{r3} = 0.85$ and $\varepsilon_u = 0.003$ were used as specified by ACI 318-56. Even so, an excellent agreement exists. The calculated values of ε_{su} are somewhat low as compared to the curves, which results from the fact that $\varepsilon_u = 0.003$ was purposely chosen as a low and safe value in the development of the 1956 Code. In this manner, the practical design criteria of the 1956 Code are confirmed by Rüsch's findings.

APPLICATION TO COLUMNS

The implications of Rüsch's sustained load design chart in terms of American column design practice were studied by comparing rectangular stress distribution calculations based on the 1956 Code ($k_{r1} = k_{r3} = 0.85$, $\varepsilon_u = 0.003$) and on sustained load ($k_{r1} = k_{r3} = 0.76$, $\varepsilon_u = 0.0067$). The difference between ultimate strengths so calculated depends on the eccentricity of loading, the proportions of the cross section, and properties of the concrete and steel used. A reduction in ultimate strength for sustained loading of $(0.85 - 0.76)/0.85 = 10.6$ percent occurs for the academic case of a concentrically loaded plain concrete column. For the heavily reinforced column shown in Fig. D, which is typical of advanced American designs for tall buildings, the reduction for concentric loading is (3569 — 3370) kips/3569 kips = 5.6 percent. This is because a major portion of the load is carried by the reinforcement, which for both calculations is assumed to be yielding at ultimate strength.

The two interaction curves for load and moment shown in Fig. D were evaluated recognizing that concrete is displaced by the compression reinforcement. Stress in the compression and tension reinforcement were both calculated by consideration of strains. The compression reinforcement was not always assumed to be yielding, which over-simplification of the 1956 Code should probably be discontinued in a future code revision.

It is seen in Fig. D that the low values of $k_{r1} = k_{r3} = 0.76$ for sustained loading, as compared to $k_{r1} = k_{r3} = 0.85$ for the 1956 Code, are compensated in some cases by the high sustained load strain, $\varepsilon_u = 0.0067$, which strain is only 0.003 by the Code. Thus, in the region of large eccentricities where strength is governed by yielding of the tension steel, sustained load calculations lead to slightly greater ultimate loads for a given eccentricity than the Code calculations. This is partly due to the fact that the compression steel is calculated to be yielding at ultimate strength for sustained loading at much greater eccentricities than for the 1956 Code, as indicated by arrows at the lower right in Fig. D.

The difference between the two loading cases, as illustrated by the two curves in Fig. D, may first be viewed in terms of normal scatter

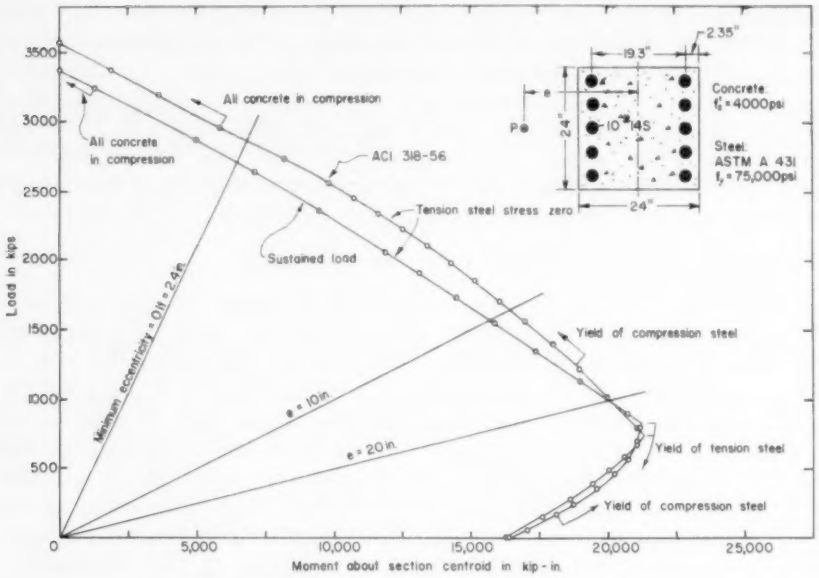

Fig. D—Strength of eccentrically loaded column

of well executed tests of reinforced concrete columns and beams, for which a coefficient of variation of at least 6 percent must be expected. This corresponds to a total scatter of about plus or minus 10 percent. Secondly, the difference may be viewed in terms of the minimum load factor of 2.0 given for columns by the 1956 Code. It does not seem unreasonable to let the load factor for the relatively unlikely case of sustained overload remain reduced to 1.8 or 1.7.

It should finally be noted that, as indicated by arrows in the top left corner of Fig. D, the case of uniform compression over the entire concrete cross section calculated by the rectangular stress distribution begins at eccentricities less than the minimum of $0.1t$ given by the 1956 Code. Hence, the mathematical discontinuity of calculations by the rectangular stress distribution occurring when $k_u = 1/k_{r1}$, that is when the neutral axis is outside the section so that the rectangular stress block covers the entire cross section, are more of academic than of practical interest. This is particularly so because calculations by the 1956 Code rectangular distribution reduces for zero eccentricity to the equation

$$P_o = 0.85 f_c'(A_g - A_{st}) + A_{st} f_y$$

which equation was originally derived from numerous tests of concentrically loaded columns during the ACI column investigation in the 1930's.

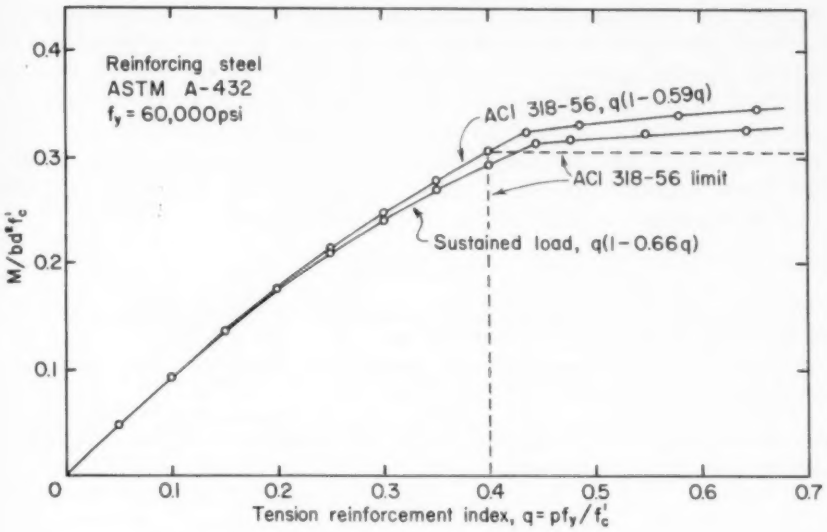

Fig. E—Strength of rectangular beams

This study of the implications of Rüsch's findings on American column design is, of course, highly incomplete. It appears to the writer, however, that the calculation methods of the 1956 ACI Code are probably reasonable for practical design purposes, even in the light of Rüsch's new findings regarding effects of sustained loading.

APPLICATION TO BEAMS

Implications of Rüsch's sustained load findings on American beam design methods were studied in a similar manner as for columns. A comparison for beams with tension reinforcement only is shown in Fig. E. It is seen that consideration of sustained loading leads to an insignificant departure from calculation by the 1956 Code methods.

CONCLUSIONS

Rüsch has presented a greatly improved and broadened base for structural concrete flexural theory. His conclusion that "it is unthinkable that practical design should involve the effect of duration of loading in detail," is certainly entirely reasonable. To this the writer would add that the use of the rectangular concrete stress distribution in everyday design practice, with which American designers have become fairly well accustomed, should not be discontinued unless it is reasonably imperative to do so. Rüsch's work has "not yet come to a decisive con-

clusion." On the basis of the limited studies presented in this discussion, it seems reasonable to anticipate that a future mature development of Rüsch's general theory can be utilized in American design practice by relatively minor adjustments of the rectangular stress distribution coefficients given the values $k_{r1} = k_{r3} = 0.85$ and $\varepsilon_u = 0.003$ in the 1956 ACI Building Code. A case of such adjustment already exists in the 1956 Code, namely, that $k_{r1} = 0.85$ "is to be reduced at the rate of 0.05 per 1000 psi concrete strength in excess of 5000 psi."

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By LADISLAV B. KRIZ*

The outstanding contributions to ultimate flexural strength theory presented by Professor Rüsch involves a determination of that strain in the extreme compressive fibers, which corresponds to the maximum moment of the internal compressive force with respect to the centroid of the tensile forces in the reinforcement. This strain is determined by assuming that the position of the neutral axis is constant, while the strain in the extreme fibers varies. This assumption is not in complete mathematical agreement with the actual observed behavior of reinforced concrete members, in which the position of the neutral axis shifts as the curvature of the member changes under increasing moment. The objective of this discussion is to examine the errors due to this simplifying assumption in qualitative and quantitative terms.

The resisting moment of a rectangular reinforced concrete cross section, Fig. F, can be expressed by the equation:

$$M = C\,jd = bd^2\,jk\,f_{ave} \qquad (3)$$

The values of j, k, and f_{ave} can all be expressed as functions of the strain in the extreme fibers. Hence, at ultimate strength in pure bending, the equation:

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$$\left(\frac{dM}{d\varepsilon_c}\right)_{\epsilon_c=\epsilon_u} = bd^2\left[jk\frac{df_{ave}}{d\varepsilon_c} + k f_{ave}\frac{dj}{d\varepsilon_c} + j f_{ave}\frac{dk}{d\varepsilon_c}\right]_{\epsilon_c=\epsilon_u} = 0 \quad (4)$$

must be satisfied provided that $M(\varepsilon_c)$ is continuous at ε_u. According to Fig. F,

$$j = 1 - k_2 k \quad (5)$$

so that

$$\frac{dj}{d\varepsilon_c} = -\left(k\frac{dk_2}{d\varepsilon_c} + k_2\frac{dk}{d\varepsilon_c}\right) \quad (6)$$

Substituting Eq. (5) and (6) into Eq. (4) yields:

$$\left[(1-k_2k)k\frac{df_{ave}}{d\varepsilon_c} - k^2 f_{ave}\frac{dk_2}{d\varepsilon_c} + (1-2k_2k)f_{ave}\frac{dk}{d\varepsilon_c}\right]_{\epsilon_c=\epsilon_u} = 0 \quad (7)$$

Eq. (7) is the criterion for the ultimate flexural strength, and therefore it is the equation from which the corresponding strain ε_u in the extreme fibers should be determined.

Keeping the value of k constant, as proposed by Professor Rüsch, is equivalent to setting $dk/d\varepsilon_c = 0$, so that Eq. (7) becomes

$$\left[(1-k_2k)k\frac{df_{ave}}{d\varepsilon_c} - k^2 f_{ave}\frac{dk_2}{d\varepsilon_c}\right]_{\epsilon_c=\epsilon_R} = 0 \quad (8)$$

The strain ε_R determined by Eq. (8) cannot satisfy Eq. (7). Hence, it does not lead to a strictly correct value of the ultimate moment. Since it does not yield the maximum value of the resisting moment, it can be concluded immediately that the error involved must be on the safe side. Nevertheless, it is desirable to examine the magnitude of the error involved.

The values of ultimate strain and ultimate moment were computed for a series of beams by two methods: one based on Eq. (7), the other on Eq. (8) which was obtained from the theory proposed by Rüsch.

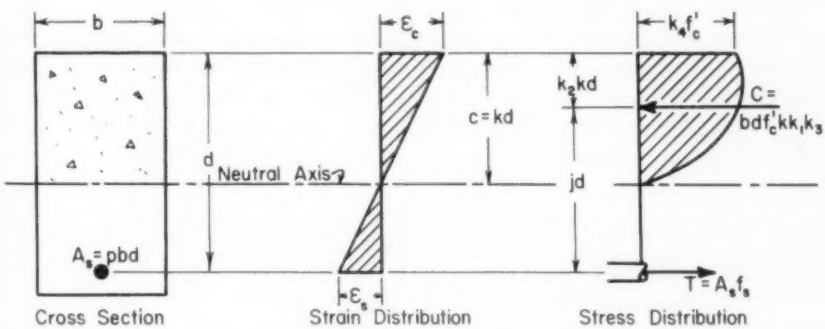

Fig. F—Conditions in a reinforced concrete beam in pure bending

Two stress-strain curves of concrete in flexure were assumed: (1) the stress-strain relationship given by Rüsch in Fig. 6 for 1 hr loading and $f_c' = 3000$ psi; and (2) a stress-strain curve obtained at the PCA Laboratories for $f_c' = 5000$ psi.[2,15] The stress-strain relationship of the reinforcement was assumed to be

$$\left.\begin{aligned} f_s &= E_s\varepsilon_s & 0 \leq \varepsilon_s \leq \frac{f_y}{E_s} \\ f_s &= f_y & \varepsilon_s \geq \frac{f_y}{E_s} \\ E_s &= 30 \times 10^6 \text{ psi} \end{aligned}\right\} \quad \ldots\ldots (9)$$

The reinforcement ratio was chosen at $p = 0.01, 0.02, 0.03, \ldots 0.07$.

The strain ε_u, which satisfies Eq. (7) was determined by calculating the value of the resisting moment for increasing values of ε_c. The value of k was also computed for each strain increment. The values of the maximum moment, M_u, and the corresponding extreme fiber strain, ε_u, were then selected from a graph of M/bd^2f_c' versus ε_c by inspection.

The values of k which satisfy Eq. (8) were likewise determined for each strain increment. To facilitate the solution of Eq. (8), the differentiation with respect to ε_c was performed, and after simplification the following expression was derived:

$$(k)_{\varepsilon_c = \varepsilon_R} = \frac{k_4 - k_1k_3}{k_1k_2(1 - 2k_2)} \quad \ldots\ldots (10)$$

The symbols used in Eq. (10) are defined in Fig. F.

The value of ε_R which satisfies Eq. (8) is that value of the strain in the extreme fibers, at which the value of k, obtained from conditions of linear distribution of strains and of equilibrium of forces, is equal to the value obtained from Eq. (10). The moment M_R is the resisting moment corresponding to the strain ε_R.

The procedure described above is illustrated in Fig. G, where two beams are considered.

The results obtained from analysis of 14 beams are presented in Table A. The values of the ultimate moments, obtained by the two methods, are close indeed, the error being less than 1 percent in all 14 beams. The results also confirm the conclusion drawn previously, i.e., the error involved in the theory proposed by Professor Rüsch is on the safe side.

The results show further that the strains at M_R are smaller than those at M_u in case of beams controlled by compression, and larger in case of beams controlled by tension.

In the rectangular beams controlled by tension, the strain ε_u is a function of the stress-strain relationship of concrete only. This is in agreement with the findings reported earlier.[11]

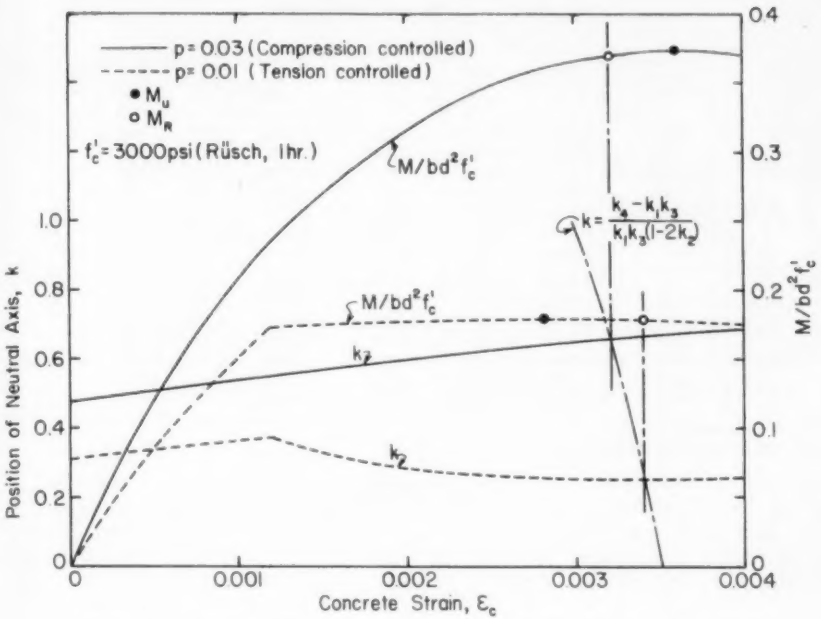

Fig. 6—Determination of ε_u, M_u, ε_R, and M_R for two beams

The errors in the calculated ultimate moments are negligibly small although the errors in the calculated ultimate strains may be considerable. This points out that the values of ultimate strain are relatively unimportant, as long as values of k_1k_3 and k_2, corresponding to the assumed value of the ultimate strain, are used in calculating the ultimate moment.

This examination of the results of the flexural theory proposed by Professor Rüsch has been limited to rectangular reinforced concrete beams. Similar comparison could be made for T-beams and for beams with triangular compression zone. The ultimate strain for T-beams given by Rüsch in Fig. 8 appears to be independent of the position of the neutral axis. The same value of ultimate strain is given for both $k = 1$ and $k = 0$. This ignores the fact that as k in a given T-section approaches zero, the neutral axis must enter the flange of the T-section, no matter how small the thickness of the flange is. Hence, for $k = 0$, the ultimate strain should be the same as for a rectangular section. Otherwise it may be expected that the observations made above regarding the conditions at ultimate strength of rectangular reinforced concrete beams, apply equally well to the T-beams and the beams with a triangular compression zone.

TABLE A — BEAM ANALYSIS

Stress-strain relationship	Percentage of reinforcement, p	Ultimate strain		Ultimate moment		ϵ_u	M_u	Controlled by
		ϵ_u	ϵ_R	M_u/bd^2f_c'	M_R/bd^2f_c'	ϵ_R	M_R	
Rüsch	0.01	0.00280	0.00340	0.1785	0.1780	0.824	1.003	T*
$f_c' = 3000$	0.02	0.00280	0.00328	0.3140	0.3126	0.854	1.004	T
1 hr	0.03	0.00360	0.00321	0.3741	0.3709	1.121	1.009	C†
	0.04	0.00350	0.00318	0.3887	0.3866	1.101	1.005	C
	0.05	0.00342	0.00316	0.3990	0.3970	1.082	1.005	C
	0.06	0.00335	0.00315	0.4063	0.4050	1.063	1.003	C
	0.07	0.00328	0.00313	0.4119	0.4110	1.048	1.002	C
PCA	0.01	0.00240	0.00287	0.1118	0.1117	0.836	1.001	T
$f_c' = 5000$	0.02	0.00240	0.00282	0.2072	0.2066	0.851	1.003	T
	0.03	0.00240	0.00276	0.2862	0.2850	0.870	1.004	T
	0.04	0.00298	0.00273	0.3227	0.3265	1.092	1.007	C
	0.05	0.00290	0.00272	0.3353	0.3333	1.066	1.006	C
	0.06	0.00287	0.00270	0.3450	0.3432	1.063	1.005	C
	0.07	0.00286	0.00269	0.3523	0.3510	1.063	1.004	C

*T = Tension controlled.
†C = Compression controlled.

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By R. G. SMITH*

Professor Rüsch and his colleagues are to be congratulated for their elucidation of the effects of creep and shrinkage on the distribution of stress and strain on concentrically and eccentrically loaded prisms.

The derivation of the stress-strain relationships for a flexural member by the method proposed by Rasch,[5] is based on the assumption that the rate of strain in any fiber remains constant. In a reinforced or prestressed concrete beam, the depth of the neutral axis, c, is *decreased* as the load is increased, so that the rate of strain in any fiber in the compression zone is not constant and at certain fibers may even be negative.

In Fig. H, c_i is the depth of the neutral axis at time t_i due to some intermediate loading; c_u is the depth of the neutral axis at time t_u when failure occurs.

Considering as a special case the strain in the fiber at depth z, it can be seen that the total change in strain in time $t_u - t_i$ is zero. It is also obvious that other fibers in the compression zone will be subjected to a variable rate of strain. To obtain a "true" correlation between the prism tests and a flexural member, it would seem necessary to take

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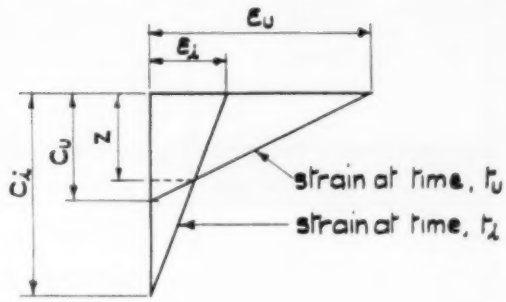

Fig. H—Strain distributions in a beam above the neutral axes at two stages of loading

into account the rate of change of the neutral axis depth so that the strain history of each fiber could be obtained. It would be interesting to know the magnitude of the error involved, especially in the case of the triangular cross section.

Professor Rüsch's "general theory" seems to be limited to the case of flexural members where there is good bond between the concrete and the reinforcement, as the assumption is made that the strain in the reinforcement is equal to the nominal strain in the adjacent concrete, plane sections beings considered to remain plane throughout the depth of the member. This would seem to preclude the application of the theory to prestressed post-tensioned members, unbonded or bonded by grouting.

Short term loading tests have been made at the University of Aberdeen on unbonded prestressed concrete beams, subjected to four-point loading. They are of rectangular cross section, 6 in. deep by 3 in. wide and are prestressed by horizontal wires located at the edge of the "middle third." Details of two typical beams are given in Fig. I and Table B. The aggregate used was $\frac{3}{8}$ in. crushed granite with river sand using a cement to aggregate ratio of 1:3.7 by weight. Concrete strains were measured by a 2-in. demountable strain gage, readings being taken along the whole length of the beam between the central load positions.

It may be seen that in the case of Beam A where only one crack developed, there is a high concentration of strain in the concrete above the crack, with a maximum value of 0.011.

In the case of Beam B where four cracks developed, the strain concentration is much lower.

TABLE B — BEAM DETAILS

Beam	Wire diameter, mm	Number of wires	Initial prestress, percent	Cylinder strength, psi	K_u	Loading time, hr
A	5	2	15	6250	0.14	1.5
B	5	4	40	6880	0.35	2.0

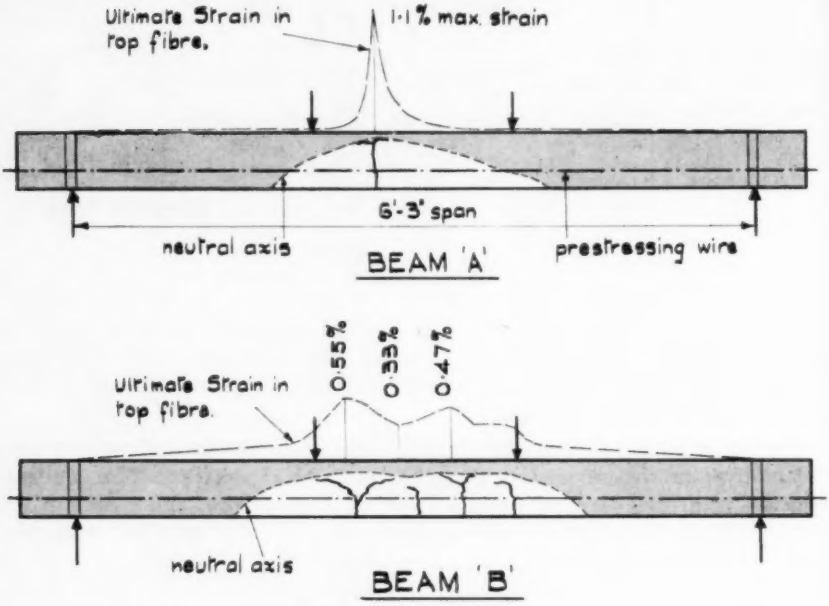

Fig. 1—Beam details

The difference in the magnitudes of the maximum strains between the two beams cannot be accounted for as a time effect, although differences in the values of K_u and the cylinder strengths may have secondary effects. Eccentric loading tests on prisms made from the same concrete and tested over approximately the same time interval had ultimate strains not exceeding 0.46 percent.[16] It seems, therefore, that the magnitude of the ultimate strain depends to a great extent on the number and spacing of the cracks in the region of constant bending moment. In the case of a reinforced concrete or prestressed pretensioned beam with good bond, sufficient cracks develop to ensure an approximately uniform depth of the neutral axis with a consequent uniform distribution of strain. Correlation of the stress-strain curves obtained by prism tests to flexural members of this type is probably sufficiently accurate. In the case of members where bond is less efficient, and particularly in the case of unbonded beams, the application of the results of prism tests can lead to significant errors in the estimation of the compressive stress distribution.

Yet another reason for the differences in the values of ultimate strain, α, etc., found by different investigators, is the effect of the gage lengths of the strain gages used. Fig. J shows the apparent ultimate strains that would have been measured for Beams A and B, by various strain gage

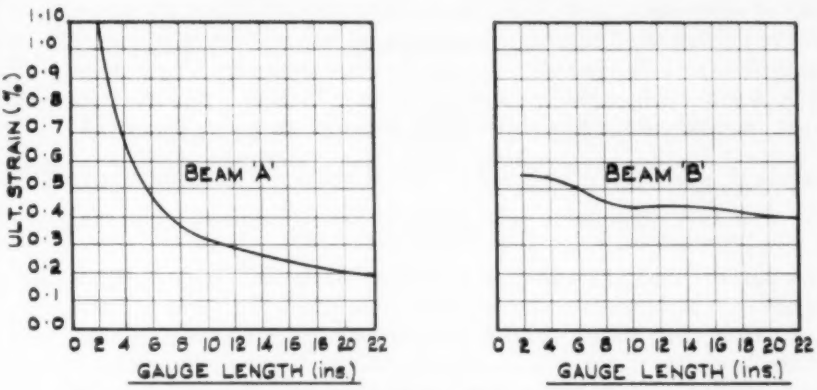

Fig. J—The effect of gage length on the apparent ultimate strain

lengths. Empirical constants used in certain theories based on experimental work using a particular gage length may in some cases be subject to errors of as much as 100 percent. The writer wholeheartedly agrees with Professor Rüsch when he says that the engineer "cannot be convinced by approximately correct results obtained on the basis of widely different assumptions."

Finally, it is well known that geometrically similar prisms of different sizes are subjected to a "scale effect," i.e., the compressive strength and standard deviation of the concrete is decreased as the size of the specimen is increased. Does Professor Rüsch think that this phenomenon is of sufficient significance to be included in a general theory?

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AUTHOR'S CLOSURE

The author appreciates the excellent comments made on his paper. The discussers added some interesting aspects and thus have increased the scope of the original work.

The results of the computations carried out by Dr. Hognestad were very satisfactory. They prove that the design procedures currently in use result in structures having a sufficient degree of safety.

A number of problems were briefly referred to which can be clarified only by some additional investigations. Mr. Smith mentioned the influence of gage length on strain measurements and pointed out that

size of specimens, shifting of the neutral axis due to creep of concrete, and imperfect bond between steel and concrete are of importance. Mr. Ali indicated the influence of compression reinforcement. The author gratefully acknowledges these stimulating contributions. However the principal purpose of his paper was only twofold:

1. To study the influence of time on strength and deformation properties of concrete.
2. To find a reasonable explanation for some discrepancies between several investigations on similar problems.

The author himself pointed out that this flexural theory still is quite incomplete and needs to be worked on further.

An extremely interesting contribution was presented by Mr. Kriz. In the author's paper the maximum moments were determined under the assumption that the position of the neutral axis remains constant. Mr. Kriz studied the possible error due to this assumption. The author regrets that he did not specially mention that this assumption was introduced as an approximation. He is thankful to Mr. Kriz for having proved that this simplification did not cause an appreciable error in the final results. In connection with this, the author would like to refer readers to a paper by Pucher* which is only little known among concrete researchers. This paper deals with somewhat similar problems.

Several discussers mentioned that it might be too unrealistic to base a design procedure on the assumption of sustained over loads. To give an exact answer to this question consideration has to be given to the theory of safety of structures, but this appears to be beyond the scope of this paper. However, even without further proof, the author still believes his assumption to be quite reasonable, i.e., that concrete at an age of 28 days might be subjected to the calculated external moments for a period of 1 day.

Finally the author wholeheartedly agrees with Mr. Ali that one of the most important tasks in concrete research will be to extend our knowledge in the theory of safety of structures.

The author would be very happy if some other laboratories and researchers will contribute to the solution of the problems necessary to improve his still quite incomplete suggestion for a flexural theory.

*Pucher, A., "Der Einfluss der Bruchstauchung des Betons auf die Tragfähigkeit von Stahlbetonbalken," *Die Bauwissenschaft* (Vienna), 1948.

Discussion of a paper by Wei-Wen Yu and George Winter:

Instantaneous and Long-Time Deflections of Reinforced Concrete Beams Under Working Loads*

By ÅKE HOLMBERG,
MURREL O. WILBURN, and AUTHORS

By ÅKE HOLMBERG†

To elucidate the interesting problems dealt with by the authors, some recent Swedish studies are reported here.

The leading manufacturer of reinforcing steel in Sweden, Halmstads Järnverks AB, makes a special high quality bar for ordinary reinforced concrete called HJS 70 ($\sigma_{YP} \approx 100{,}000$ psi) which are plain bars with end anchorages. When introducing them about 10 years ago they wished to give some advice on how to predict beam deflections.

On the basis of scanty knowledge from one long-time full-scale test (described by the writer in *Tekniska Föreningens i Finland Förhandlingar*," Helsinki, 1952) and from theoretical considerations the following recommendations were given.

Deflection shall be calculated with cracked section assumed and for high grade concrete with a modular ratio according to the table below:

Conditions of loading	Modular ratio
Short-time load, fixed ends	10
Long-time load, fixed ends	20
Short-time load, free ends	30
Long-time load, free ends	40

Thus the influence of creep, as well as of a shrinkage, being larger in the upper portions of the beam, should have been taken into consideration.

When starting to manufacture a standard series of precast concrete beams with HJS 70 bars and a concrete having a cylinder compressive strength about 5500 psi. A-Betong, a Swedish producer of precast units, applied these recommendations in their calculations. The beams in question are described by the writer ("Die Montagebauweise mit Stahlbetonfertigteilen," II, Internationaler Kongress, Dresden, 1957).

*ACI Journal, V. 32, No. 1, July 1960 (*Proceedings* V. 57), p. 29. Disc. 57-2 is a part of copyrighted Journal of the American Concrete Institute, V. 32, No. 9, Mar. 1961 (*Proceedings* V. 57).

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During the summer of 1960, about 50 freely supported beams 1 to 3½ years old were inspected. All of them were interior beams. Total length varied between about 30 and 65 ft.

Within the limits of accuracy for the observations, observed and calculated deflections coincided. The great influence of age mentioned by the authors thus was not verified.

In other respects there is no great difference between the conclusions drawn from the two investigations. So one should now have a simple method to predict beam deflections when using high grade concrete and high grade plain reinforcing bars.

By MURREL O. WILBURN*

The three factors that can produce long time sag or increase dead load deflections in simple span reinforced concrete beams, in order of importance, are: drying shrinkage of concrete, creep, and plastic flow. These three factors are usually lumped into a single term and the ultimate dead load deflection is estimated by multiplying the normal elastic dead load deflection by some factor based more or less on experience. The author's Methods C and D fall into this category and Tables 2 and 4 illustrate that there is considerable guess work in the use of even the best experience factors to estimate the ultimate dead load deflection of a particular reinforced concrete beam. In estimating the ultimate dead load deflection of a reinforced concrete beam each of these factors, shrinkage, creep, and plastic flow, should be considered separately.

Drying shrinkage of concrete changes the physical shape of the beam.[13] Drying shrinkage of concrete increases the dead load deflection of simple spans and also affects the dead load stresses somewhat when the beam has compression reinforcing steel. In the continuous span it can produce a considerable change in dead load stresses.[14]

Creep, when separated from shrinkage, is a delayed deformation similar to elastic deformation and is produced by the same stresses that produce elastic deformation. At working stresses creep is proportional to stress, thus its effect on deflection can be estimated by an appropriate reduction in the modulus of elasticity used for concrete. It is believed that the effect of creep on the deflection of reinforced concrete beams is usually greatly exaggerated or overestimated in structures such as highway bridges carrying short duration live loads with maximum stresses within allowable working limits.

Plastic flow in concrete is that deformation which occurs at high stress (above 70 percent of ultimate), the concrete acting as a viscous solid. Plastic flow deformations are not proportional to stress. Plastic flow, if such ever occurs in a reinforced concrete member, produces plastic hinges at points where plastic flow occurs. If plastic flow ever

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occurs in a continuous system, there is a loss of continuity within the system and the stresses within the system are changed.

It is believed that drying shrinkage of concrete is the major contributing factor to the long time increase in dead load deflections of simple reinforced concrete bridge spans. It is now known that the concrete shrinkage factor in thin concrete sections such as floor slabs of a T-beam bridge span is at times as high as 0.0006. This factor varies with the concrete mix, climate, etc., but is primarily affected by the water content of the concrete member at a particular time. It has been demonstrated[13] that a concrete beam with reinforcing steel on one side which was cast straight and allowed to warp through the drying shrinkage of the concrete will become straight again when the concrete of the beam becomes saturated with water. Therefore, it is probable that the dead load deflection of a reinforced concrete bridge span will change between a prolonged wet season and a prolonged dry season.

In calculating the effect of shrinkage on deflection, it is assumed that maximum concrete shrinkage occurs at the top surface of the floor slab and that shrinkage has no effect or is zero in the plane of the tension reinforcing steel. This is true through lengths of the beam with sufficient dead load and live load bending moments to stress the concrete beyond its ability to carry tension stress. Through these lengths the concrete will crack and the reinforcing steel must carry the entire tension load. In some beams there is probably a short length at each end of the span where tension stresses produced by dead and live loads are not sufficient to crack the concrete. The shrinkage in the bottom of the beam will increase tension in the concrete and may add some compression to the reinforcing steel with the result that portions of the bottom flange will not lengthen quite as much as the elastic theory predicts. However, changes in fiber lengths in these short portions of the beam, which are always small, have a small effect on the deflection of the beam at the center of the span.

It is also assumed that the shrinkage has a straight-line variation through the depth of the beam from a maximum at the slab surface to zero at the center of the reinforcing steel. This is not quite true. It is probable that the concrete in the stem of the beam in the vicinity of the junction of the stem with the floor slab does not dry out enough to produce the amount of shrinkage assumed by the straight-line theory. This means that the concrete in this area of the stem will be carrying more compression stress than the flexure theory predicts. In other words, the reinforced concrete stem of the T-beam may be considered to be acting with the compression reinforcing steel to carry a portion of the dead load bending moment at one stage during the adjustment of stresses caused by shrinkage.

In the center portion of the span where the dead load bending moments are high, the adjustments of stresses which, of course, effects deflection are quite complicated. To understand these actions it is necessary to consider them in steps, even though they occur more or less simultaneously. Some shrinkage has occurred in the concrete before the span is made self-supporting but we will consider the first stage to be with the span supporting the dead load in accordance with the elastic theory. No concrete shrinkage has occurred. The concrete in the compression flange is carrying its normal load. The longitudinal reinforcing steel in the floor slab, which is usually called distribution or shrinkage reinforcement, is carrying its portion of the compression load and the tension reinforcing steel in the bottom flange is carrying the tension load.

Now we will allow the full concrete shrinkage to occur. If there was no longitudinal reinforcing steel in the compression flange and there was a straight-line variation of shrinkage through the depth of the beam, there would be no change in the stress pattern of the beam. We could calculate the warpage or sag produced by shrinkage and add this to the elastic deflection and the problem would be solved. But this longitudinal compression reinforcing steel, even though its area is usually small, changes the picture. Concrete shrinkage in the compression flange relieves the concrete of carrying any portion of the compression load temporarily. The beam becomes a "steel" beam made up of tension and compression reinforcing bars only which carry the tension and compression loads produced by a portion of the bending moment. This steel beam carries a sufficient portion of the bending moment to produce elastic shortening in the compression reinforcing steel equal to the shrinkage in the concrete compression flange, then the concrete comes back into action. Through lengths of the beam with low dead load bending moments, near the ends of the span, the elastic shortening of the compression reinforcing steel probably never equals the shrinkage of the concrete, so the dead load compression stresses in these portions of the beam are carried entirely by the reinforcing steel. We can calculate the moment of inertia of this steel beam easily and determine the amount of bending moment it takes to shorten the compression reinforcing steel an amount equal to the concrete shrinkage. If the concrete shrinkage factor is 0.0006, the stress in the compression reinforcing steel to produce this amount of elastic shortening is $0.0006 \times 29{,}000{,}000 = 17{,}400$ psi. Having calculated the moment of inertia of the steel beam, we now calculate the bending moment required to produce a stress of 17,400 psi in the compression steel and also calculate the tension stress f_{s1} in the tension steel.

The concrete compression flange has now come into action and the beam will act in accordance with the normal elastic theory except for

the slight fallacy of considering a straight-line variation of shrinkage through the depth of the beam, to carry the remainder of the dead load bending moment. The tension reinforcing steel will now have an additional tension stress, f_{s2}, produced by the remaining dead load bending moment. The total dead load stress in the tension reinforcing steel will equal $f_{s1} + f_{s2}$, which will be higher than given by the normal application of the homogeneous transformed section theory. The concrete in the compression flange will be carrying less stress than given by the methods normally used for calculating stresses in reinforced concrete beams.

It is now possible to calculate the ultimate dead load deflection of the span by using the conjugate beam method. The portion of the bending moment diagram carried by the "steel" beam is divided by the moment of inertia of the steel beam and the remaining portion of the bending moment diagram is divided by the moment of inertia of the transformed reinforced concrete beam. The sum of the bending moments produced at a particular point in a conjugate beam loaded with each of these two M/I diagrams divided by the appropriate modulus of elasticity ($E_s = 29{,}000{,}000$ and $E_c = 1000\ f_c'$) is the ultimate dead load deflection, δ_{ct}, at that point.

REFERENCES

13. Miller, Alfred L., "Warping of Reinforced Concrete Due to Shrinkage," ACI JOURNAL, V. 29, No. 11, May 1958 (*Proceedings* V. 54), pp. 939-952. Disc. V. 30, No. 6, Part 2, Dec. 1958 (*Proceedings* V. 54), pp. 1393-1402.

14. Wilburn, Murrel O., "Nonelastic Deformations in Continuous Concrete Structures," *Public Roads*, V. 30, No. 8, pp. 191-196.

AUTHORS' CLOSURE

The interesting contribution by Mr. Holmberg is, unfortunately, too brief and lacking in detail to permit proper evaluation. In addition, and more important, the type of construction with which it is concerned is clearly outside the range of validity of the paper. This is so because all tests cited in the paper, and from which the various methods of deflection calculation were developed, refer to members reinforced with straight bars deformed to the deformation requirements of ASTM A 305. In contrast, the reinforcement described by Mr. Holmberg consisted of plain bars of high strength furnished with end anchorages. It is reasonable to assume that beams so reinforced, particularly simple beams, act more like flat arches with uniform steel stress over most of their span, than as beams in the ordinary sense. This evidently leads to larger deflections, a factor which is confirmed by Mr. Holmberg's unusually high value of the modular ratio for short time loads of beams with free ends. It is equally clear that less of this arch action is present in fixed-ended beams with such reinforcement, a fact which is in-

directly accounted for in the more normal value of the modular ratio proposed by Mr. Holmberg for fixed beams.

Mr. Holmberg suggests from one laboratory test and from field observations that he could not verify the extent of the influence of duration of loading reflected in the authors' data. In view of lack of details of Mr. Holmberg's evidence it is not possible to comment on this observation. For instance, Fig. 3 clearly shows that time effects are greatly reduced by the presence of compression reinforcement, but it is not known whether such reinforcement was present in Mr. Holmberg's beams.

To summarize, Mr. Holmberg's brief contribution presents interesting evidence on a type of construction which is outside the range of the original paper.

Mr. Wilburn, in his interesting contribution, ascribes most of the long-time deflections, at least in simple bridge spans, to drying shrinkage. His ideas are set forth in greater detail in Reference 14. Mr. Wilburn is quite right in stating that the methods proposed by the authors for predicting long-time deflections do not differentiate between the effects of shrinkage and of creep. In fact, this is explicitly stated in several places in the paper and it is said that the primary purpose of these intentionally simplified methods was to serve as practical engineering tools. It might be added here that the chief application the authors had in mind was to buildings rather than bridges. The innumerable beams and slabs of moderate span which occur in buildings of any significant size would make elaborate, special methods of long-time deflection calculation prohibitive while such calculations may be quite in order for the small number of long-span main girders in bridges.

Mr. Wilburn's contentions that the major part of long-time deflections is caused by shrinkage rather than creep seems somewhat at variance with the elaborate test data analyzed in the paper, at least for the type of members for which experimental results were available. The deflections caused by shrinkage depend entirely on the asymmetry of the reinforcement; beams with equal reinforcement along top and bottom faces should show no deflections caused by shrinkage. The evidence presented in Fig. 3 and Table 2 shows that the presence of compression reinforcement indeed does reduce long-time deflections considerably. At the same time it will be observed that in specimens with equal tension and compression reinforcement, long-time deflections still amounted to 80 percent and more of the instantaneous deflections. In this case these long-time deflections are evidently caused solely by creep, with no contribution from shrinkage. It should be noted, moreover, that long-time creep deflections in compression reinforced beams would be smaller than in beams without such reinforcement even in the complete absence of shrinkage. This is so because the creep deformations in the concrete tend to reduce the concrete compression stresses

at the expense of causing increased compression stresses in the steel. Since creep is reasonably proportional to the intensity of the concrete stresses, this stress redistribution reduces the final creep strains and the consequent long-time deflections in compression reinforced beams.

To be specific, it will be recalled that Method C for predicting long-time deflections employs an effective modulus for concrete, an approximate approach which is basically consistent with the nature of creep deformations which are proportional to stress, but is entirely inappropriate for shrinkage deformations which are independent of stress. From a numerical analysis of the results it is noted on p. 36 that "... the effect of compression reinforcement is somewhat overemphasized . . . in this simplified method." That is, at least as far as these tests are concerned, the presence of compression reinforcement was found to reduce the deflections to a somewhat smaller extent than computations on a linear creep basis would lead one to expect. If shrinkage were a major factor, one should find the exactly opposite situation.

To be sure, shrinkage and its effects depend to a major extent on curing conditions, humidity regime, and similar factors. It is believed that the conditions of the majority of the large number of tests which the writers have evaluated correspond reasonably well with representative conditions in concrete building construction. In the field of bridges, to which Mr. Wilburn's approach is primarily addressed, conditions are different and there the effects of shrinkage may conceivably be more severe than could be deduced from the experimental evidence on which the authors' methods are based.

Discussion of a paper by John E. Bower and Ivan M. Viest:

Shear Strength of Restrained Concrete Beams Without Web Reinforcement*

By GEOFFREY BROCK, C. W. THURSTON, and AUTHORS

By GEOFFREY BROCK†

The tests reported in this paper were examined with special interest by the present writer. They can be compared with the predictions of his recent JOURNAL paper on the effects of shear.‡ In that paper the ultimate moment and shearing force at the critical section of a rectangular beam were related by the use of parameters p/p_o and M/Vd. The relation was given in chart form as Fig. 12.

The results of the author's Series II tests can be compared directly with the chart (Fig. A). These tests covered the effect of varying M/Vd while p/p_o was held constant. (The computed values of p/p_o actually varied from 0.46 to 0.58 with about a mean value of 0.51. These variations are relatively insignificant and have been neglected.) Fig. A shows that the results followed the pattern associated with the shear-bond type of failure. The actual failing moments were rather lower than those predicted from the writer's tests on reinforced plaster beams. This discrepancy could mean that the bond behavior of the kind of reinforced concrete beams used in the present investigation is weaker than that of

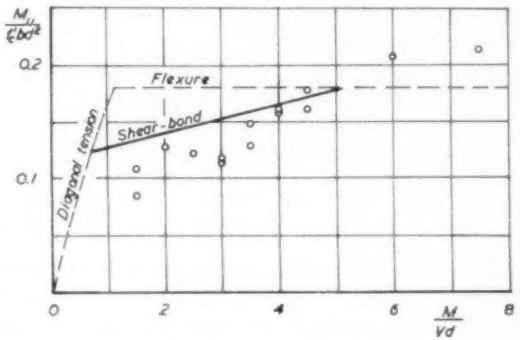

Fig. A — Tests of Series II compared with chart based on reinforced plaster beam tests (p/p_o has been assumed to be 0.5)

*ACI JOURNAL, V. 32, No. 1, July 1960 (*Proceedings* V. 57), p. 73. Disc. 57-4 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 9, Mar. 1961 (*Proceedings* V. 57).

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‡Brock, Geoffrey, "Effect of Shear on Ultimate Strength of Rectangular Beams with Tensile Reinforcement," ACI JOURNAL, V. 31, No. 7, Jan. 1960 (*Proceedings* V. 56), pp. 619-638.

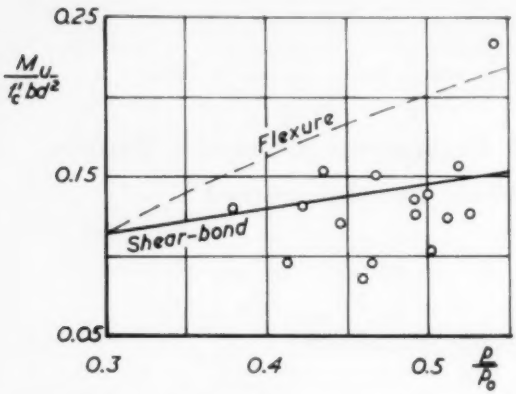

Fig. B — Tests of Series I compared with values predicted from reinforced plaster beam tests ($M/Vd = 2.50$)

the reinforced plaster beams. On the other hand, it has been difficult to judge whether the values of f_y given in Table 3 are the correct ones from which to calculate p_o ($p_o = 0.456\, f'_c/f_y$). Being uncertain of the exact shape of the stress-strain curve for the reinforcement, the writer has some doubts.

The author's Series I beams, neglecting the four of Ib, vary p/p_o while M/Vd is constant at 2.50. These require a comparison of the kind given in Fig. 13-16 of the writer's paper.* This is given as Fig. B. Again the results fall into the shear-bond pattern and once more the observed moments of resistance are on average slightly lower than those given by the reinforced plaster beams.

It is apparent that there is a great deal of random scatter in the resistance of beams failing in shear-bond. It would seem preferable to accept this, rather than to attempt to evade it by working in terms of the diagonal cracking load. The practicing designer will, in any case, wish to avoid the shear-bond mode of failure altogether. This can, of course, be done by using conventional web reinforcement in the form of stirrups. The writer has recently collected some evidence that it can also be avoided by a more realistic detailing of the main reinforcement, in which the bars are not all carried to anchorage at the same level in the beam. Some of the pioneers of reinforced concrete, notably Cottacin, seem to have been mindful of this point in preparing their designs.

By C. W. THURSTON†

This paper presents lucidly and knowledgeably both the results and analyses of a well planned series of tests. The consistency of the test data is a measure of the authors' care and control during their investigations. Their analyses are based empirically on the parameters gen-

*Ibid.

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TABLE A — COMPARISON OF SHEARS AT INITIAL DIAGONAL TENSION CRACKING

Effective shear span, in.	Number of beams	Average shear, V_c, kips	Scatter of shears averaged, V_c, kips		Percent scatter of shears averaged
			High	Low	
18	2	14.1	14.3	13.9	± 1.4
24	5	11.0	11.1	10.9	± 0.9
30	15	9.9	11.0	8.8	± 11.1
36 and greater	9	8.9	9.6	8.2	± 7.9

erally accepted by most American investigators studying the same problem.

All beams were planned to have the same steel ratio, concrete strength, and effective depths (top and bottom). As cast, all measured effective depths differed from the nominal by no more than about 4 percent and the greatest variation of the bulk of the reported concrete strengths from their average of 3330 psi was about 10 percent. Hence, in the discussion which follows, p, b, d, and f_c' are not considered variables.

From Fig. 1 and Tables 1 and 5 (noting that in Table 1 the heading of the right hand column should read P_2/P_1) the shears in each effective shear span at the initial diagonal tension cracking load can be calculated. These have been averaged and summarized in Table A. Three of the 34 beams tested have been omitted. Beam IIA-7 was reported to have had no diagonal tension cracking. Beams IA-3b and IA-5a had excessively high and low values, respectively, which were not corroborated either by the results from their companion specimens or by the trends of their neighbors.

Table A shows that for the given type of loading (concentrated loads) and beam section (p, b, d, and f_c'), the shear at initial diagonal tension cracking apparently depends on the effective shear span. This dependency may be expressed, of course, as a function of M/V, as in Eq. (4). Such a conclusion is not technically in opposition to that of the authors in their summary "that the length of the shear span ... (has) no direct effect on the diagonal tension cracking load." The difference between the two conclusions might bear emphasis, however, since the test results were presented to substaniate earlier findings, when, in fact, by themselves they primarily suggest a differing viewpoint. The writer tends to agree with the authors, but is not fully convinced, and feels that further analyses are necessary. Beams with other sizes and shapes may confirm or modify the findings of this paper, and the presentation of corresponding analyses for beams subjected to uniform loadings is earnestly desired.

Similarly, the writer is concerned with the authors' conclusion that "the diagonal tension cracking load should be considered the ultimate

design load for beams without web reinforcement." It has been established by several investigations that the initial diagonal tension cracking load is the ultimate load for beams with relatively high ratios of effective shear span length to effective depth (a/d). In this series of tests, seven of nine beams having effective shear span lengths 36 in. or greater, and failing in shear, did so at the initial diagonal tension cracking load. On the other hand, however, only three of 24 beams having effective shear span lengths under 36 in. collapsed at the initial diagonal tension cracking load. The average of the reserve strengths of these 24 beams beyond their initial diagonal tension cracking loads was 33 percent, and studies of other investigations have shown similar reserve strengths for beams with relatively low a/d ratios. Even greater significant reserve strengths exist for beams subjected to uniform loadings. Unless such reserve strengths can definitely be shown to be haphazard and unreliable, should they not be acknowledged in an ultimate load formula?

Somewhat related to the same problem is the choice of suitable descriptions for the various modes of failure. The authors have elected to identify all diagonal tension failures with the same symbol without distinguishing whether they occur immediately at the initial diagonal tension cracking load or at some higher reserve strength value. Contrasted with these are the "shear-compression" failures, which, by definition must occur at a higher load than that causing the initial diagonal tension crack. The writer feels that the basic distinction should be predicated on whether or not the beam survives the initial diagonal tension crack. An additional notation could then be included to designate the type of secondary failure (if any).

AUTHORS' CLOSURE

The test data presented by the authors as well as numerous other data reported in technical literature over the past several years have shown that shear affects the strength of a reinforced concrete beam through diagonal tension cracking. If a diagonal tension crack forms in a beam without web reinforcement, the beam either fails at the load causing the diagonal crack to form or sustains additional load. The diagonal tension cracking loads have been found to follow a consistent pattern and were correlated by several investigators with a reasonable degree of success. On the other hand the additional strength beyond diagonal cracking has been found to be erratic and attempts to correlate it to failure loads have not been successful thus far.

For these reasons the authors consider it desirable to provide web reinforcement in any beam in danger of diagonal tension cracking or, in other words, the authors suggest that for design purposes the diagonal tension cracking load should be considered the ultimate design load for

beams without web reinforcement. It is interesting to note that, even though both discussers expressed concern about this statement, neither of them proposed a reliable method for utilizing this excess strength. Furthermore, Professor Brock emphasized the scatter in the failure loads and suggested that the designer will want to avoid such failures by, for example, the use of web reinforcement.

Some of the modes of failure were designated by the authors as diagonal tension failures, while Professor Brock prefers to designate them as shear-bond failures. The preference is largely a matter of semantics as in both cases the mathematical models must fit the same test data. This is particularly so since the test data published thus far have not shown conclusively what is the primary cause of such failures.

Professor Thurston also raised a question of terminology. The authors recall that less than a decade ago the terms diagonal tension failure and shear failure were used interchangeably. Now that the problem of shear is known better, we clearly differentiate between the two terms. The authors believe that further terminology may be needed to describe accurately the failures particularly after the formation of the diagonal tension cracks.

The authors' use of the term "shear-compression failure" seems to need clarification. The term was used when a beam failed at the location of a diagonal tension crack that formed at a lower load. That is, it was used when the beam failed at a load greater than that causing the formation of *this particular* crack rather than "... at a higher load than that causing the *initial* diagonal tension crack."

Probably the most intriguing point was raised by Professor Thurston in connection with the effect of M/V. It is apparent from the test data presented by the authors that the shear span, representing physically the portion of the beam between two adjacent concentrated loads, is not a significant parameter in relation to the diagonal tension strength. That leaves only the effective shear span and the ratio of M/V. Both of these terms are abstractions and, furthermore, the following equation applies:

$$\text{Effective shear span} = M_{max}/V$$

The real question is then not between the effective shear span versus M/V but in the choice of M. Is it the maximum moment or the moment at some other section? The authors agree with Professor Thurston that tests with uniform load are desirable to clarify this point. It may be of interest to note that studies of results published recently* and from investigations still in progress suggest that the M/V at the section of diagonal tension cracking is the important quantity.

*DeCossio, Roger Diaz, and Siess, Chester P., "Behavior and Strength in Shear of Beams and Frames Without Web Reinforcement," ACI JOURNAL, V. 31, No. 8, Feb. 1960 (*Proceedings* V. 56), pp. 695-736.

The stress-strain diagrams of all reinforcing bars used in this investigation were characterized by a distinctly flat plateau at the level of the yield point. The last column in Table 1 should be headed P_2/P_1.

The authors wish to thank the discussers for their stimulating contributions that are most helpful in clarifying the concepts advanced in the paper.

Discussion of a paper by Lewis H. Tuthill:

Conventional Methods of Repairing Concrete*

By I. LEON GLASSGOLD, M. J. HAWKINS, M. R. SMITH, and AUTHOR

By I. LEON GLASSGOLD†

In any discussion of the best techniques and procedures for repairing concrete, there is bound to be a difference of opinion among the practitioners of the art. Except for the references to shotcrete which will be discussed later, the author has provided much sound and pertinent information in his paper.

In the section on bonding the author mentions a "twilight zone" of deterioration. The writer believes this soft or weakened strata is a "fatigue" zone due to the alternate freezing and thawing and/or chemical action of water absorbed into the capillary pores and hair cracks in the existing concrete surface. This twilight zone has undergone sufficient cycles of fatigue stress to reduce its cohesive qualities and compressive strength, but not enough to consider it completely deteriorated. The big question with respect to the twilight zone is: how far does it penetrate into the concrete, and consequently, how far does one chip? Experience dictates the answer, especially where the entire thickness of concrete is of poor quality. The writer feels that the minimum thickness of repair should be 1½ in. to provide sufficient coverage for the reinforcing. In those cases where deterioration has been started by corrosion of the reinforcing, chipping should penetrate behind the reinforcing bars so they can be bent back from the existing concrete surface and also provide better bond for the repair.

For concrete repair, conventional concrete and hand-placed mortar are not suitable substitutes for pneumatic mortar or shotcrete. Conventional concrete is most suitable where the location and depth of the repair make its use a more convenient and economical procedure. The use of hand patching is only justified where the area to be repaired is so small as to make it uneconomical to use shotcrete; otherwise, hand patching, at best, is inferior to shotcrete from both a structural and long range economical standpoint.

*ACI JOURNAL, V. 32, No. 2, Aug. 1960 (*Proceedings* V. 57), p. 129. Disc. 57-6 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 9, Mar. 1961 (*Proceedings* V. 57).

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The necessity for sandblasting a concrete area after chipping is questionable. If contamination of a chipped area does occur and cannot be cleaned by an air-water blast it is better to rechip the area, since sandblasting would only surface clean and not reach into the pores of the concrete. In addition, wet sandblasting tends to close these pores, thereby reducing the bonding ability of the concrete surface.

The writer agrees with the author that better bond is obtained if "joint surfaces are approaching dryness when the new material is placed on them." In shotcrete work, the writer has found that excellent bond has been achieved whether or not the surfaces were given a preliminary wetting. In fact, if the pores and voids of the base concrete are completely full of water, it is conceivable that these openings cannot absorb additional water and free cement particles (from the shotcrete) and thereby enhance the chemical and mechanical bond. In some cases, highly absorptive surfaces such as old, dry salmon brick absorb all the water in the shotcrete application almost instantaneously, thereby developing hair cracks and causing "alligatoring" of the shotcrete layer. There is no general answer as to the amount of wetting water required for the development of good repair bond. However, too much water can be detrimental to bond and only enough should be present in the base material to allow the fresh repair to set, hydrate, and form its matrix without shrinkage.

The writer must take serious objection to some of the author's comments on the limitations and disadvantages of shotcrete repair of concrete structures. The author leaves considerable doubt in the mind of the reader of the efficacy and usefulness of shotcrete repair techniques. He also has used previously printed ACI information which in many respects is contrary to the writer's experience and practice. In fact, many successful shotcrete applications would not now be in existence if the author's thesis were followed explicitly. It is true that the author based his conclusions on shotcrete's limitations, in the main, on ACI 805-51* and the writer should quarrel with it rather than with the author. Be that as it may, the writer questions the author's implications and conclusions as to shrinkage, cracking, density, porosity, resistance to freezing, and other disadvantages of shotcrete. In general, shotcrete has been used successfully for those very applications that ACI 805-51 and the author indicate might lead to trouble. Relatively thin layers of shotcrete have successfully waterproofed leaking reservoirs and other concrete structures. It has protected steel pipe and had excellent service under the most adverse conditions of usage. There have been unsuccessful applications, but they have been few in number and in all probability could have been avoided if proper application techniques and/or materials had been used. In other words, some of the properties ascribed to

*ACI Committee 805, "Recommended Practice for the Application of Mortar by Pneumatic Pressure (ACI 805-51)," ACI JOURNAL, V. 22, No. 9, May 1951 (*Proceedings* V. 47), pp. 709-720.

shotcrete by ACI 805-51 and the author do not seem to apply to actual shotcrete practice.

There is sufficient experimental evidence indicating that the absorption of water by shotcrete is less than ordinary concrete. Also, the specific gravity and strength of shotcrete are greater than ordinary concrete thereby resulting in a denser material. In addition, shotcrete utilizes only sufficient water to effect proper hydration, and consequently does not develop capillary pores to the same degree as concrete. By the nature of its application shotcrete tends to produce greater density because the impinging particles of sand in the mix tend to naturally tamp each succeeding layer of shotcrete. There is also formed at the repair surface a thin layer of fine, rich, dense mortar which in itself is highly waterproof and tends to increase bond. It is, therefore, hard to understand how shotcrete is more porous than concrete and is not satisfactory for coating of structures exposed to the weather or water, etc. The writer's experience also indicates that if shotcrete is designed for a specific application, utilizes correct materials, and is well placed, it will not exhibit an excessive number of hair cracks due to shrinkage. Cracking will develop, however, where the base material is cracked and reflects through the shotcrete.

Summing up, the writer feels that his experience and that of other responsible engineers* specializing in shotcrete applications is contrary in many ways to the practice as espoused by ACI 805-51. In addition, the Portland Cement Association bulletin on shotcrete contradicts some aspects of the ACI Standard.

Why then is there so much conflicting evidence published on the merits of shotcrete? The writer does not profess to know the answer, but suspects the lack of uniform testing procedures. He also feels practice is far ahead of formal experimentation, and with such an important engineering tool, the opposite should be true. With the use of shotcrete being expanded every day, the time has come to separate fact from myth and it behooves the American Concrete Institute to be the initiating agency.

By M. J. HAWKINS†

Having worked with Mr. Tuthill for many years where emphasis was placed on good concrete repairs when necessary in the construction program, I feel qualified to discuss his ACI article. Since I am aware of three of the five photographs in his article, I particularly wish to elaborate on the conditions presented by them.

The material presented in Mr. Tuthill's article is well known to Bureau of Reclamation construction forces. The repair methods outlined

*Stubbs, Frank W., Jr. (editor-in-chief), *Handbook of Heavy Construction*, McGraw-Hill Book Co., New York, 1959, Chapter 11—"Pneumatically Applied Concrete" by Charles W. Sharp, pp. 11-1 to 11-18.

†Member American Concrete Institute, Chief, Materials Engineering Division, U.S. Bureau of Reclamation Trinity River Project, Lewiston, Calif.

in his 1960 article follow closely those shown in the repair instructions written by Mr. Tuthill in 1947 with the title "Procedure for Repair of New Concrete and Restoration of Old Concrete." The 1947 repair guide has been used by Bureau of Reclamation construction forces for many years and concrete repairs following the repair guide have an excellent performance background. (This guide was later included as Chapter 7 in the USBR *Concrete Manual.*)

Inspection of concrete repairs generally has demonstrated without doubt that it is possible to make concrete repairs that will be equal in quality to the sound concrete adjacent to the repaired areas. One of the principle items stressed in Mr. Tuthill's report has been the need for proper clean-up of concrete surfaces to be repaired before repair materials are placed. Lack of preparation of the underlying concrete surface resulted in poor bond shown by the photograph on p. 131. In this particular case the contractor without approval placed portland cement plaster on the basement walls of an operation and maintenance building to cover defective concrete areas. At the time portland cement plaster operations were underway on another part of the building. The contractor's attempt at this make-shift plaster job resulted in having to remove the applied materials at his own expense. In the photograph the hand saw was used to show the lack of bond.

The picture shown on p. 132 illustrates a lack of concrete consolidation, or in other words, "a rock pocket." Rock pockets generally occur in walls and elsewhere where vibration is inadequate or there has been separation. Mr. Tuthill's report might well have included the method used to place repair concrete in walls. Because of the excellent performance record for the method outlined in his 1947 repair guide, I am taking the liberty to include the detailed procedure. The repair procedure included with this discussion (Fig. A) is identified as Fig. 8 in the 1947 repair guide and shows in detail forms for concrete placement in walls. When a wall face is exposed to view the chimney or casting head is removed; however, on walls to be backfilled the chimney is usually not removed. It may be of interest to mention that this wall repair method was developed in 1942 and 1943 during construction of concrete ships.* As resident concrete engineer at the Savannah, Ga., shipyard during this period, it was possible for me to check the effectiveness of this repair method. The hydrostatic testing of each compartment in the tanker type concrete ships subjected each repair to a water-tightness test with a water head extending 8 ft above the deck level. My records show that 98 percent of more than 200 repair areas extending through the wall sections of the first hulls cast withstood the hydrostatic water-tightness test.

The unsightly repair shown in the photograph on p. 136 illustrates a condition that field men must make a special effort to control. The

*See Tuthill, Lewis H., "Concrete Operations in the Concrete Ship Program," ACI JOURNAL, V. 16, No. 3, Jan. 1945 (*Proceedings* V. 41), p. 171.

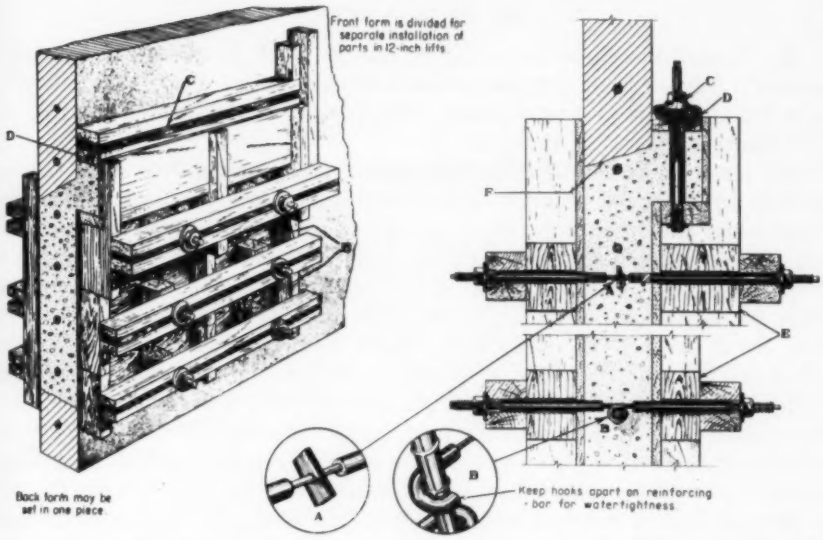

Fig. A—Details of forms for concrete replacement in walls (Fig. 8, USBR Laboratory Report No. C-335). By use of anchor bolts, these front forms may be used for replacements in the surfaces of massive concrete structures

A and B are used as shown at the center of the replacement and she-bolts are attached which may later be removed and the hole filled. Bolts around edge of patch are tight at start, but center bolts are left fairly loose. While the patch is being revibrated and the capping plate wedged or bolted down tightly, the center bolts are tightened, forcing the compacted concrete up against top of hole and capping plate, and thus counteracting settlement of the concrete. Be sure tie bolts are tightly caulked through forms to prevent mortar loss; also that joints in forms, between lifts, and between forms and concrete surface are tightly caulked for the same reason. Where it is not practicable to use the wedges for forcing the capping plate down the method using vertical bolts, shown in the section view, can be used.

C is the nut or wedges used to force capping plate down, thus compressing concrete in forms.

D is a capping plate filling chimney and acting as a piston in compressing the fresh concrete in the forms and should be tightened downward during final revibration.

E are compression blocks between waler and form sheeting to transmit squeeze of tie bolts to concrete.

F is the top of the opening being replaced which slopes upward about 3:1 toward the side from which the new concrete is placed.

mortar used to repair the construction joint was made by the contractor with a different brand cement from that used during the original concrete placement and with no effort to match the colors. The dark colored repair areas have now withstood 10 years service and are in excellent condition but still have an unsightly appearance. In some instances, inspection may require more than usual attention to be sure the contractor will obtain repair concrete of a color similar to that in surrounding concrete. In this case Mr. Tuthill has presented a field condition that merits the full attention of our inspection forces if we are to obtain concrete structures with the best possible appearance.

In closing I wish to commend ACI officials for publishing the several articles on concrete repairs in the same issue of the JOURNAL. Our August JOURNAL provides our members with a useful, practical reference library on the subject of repairing concrete.

By M. R. SMITH*

Mr. Tuthill has done his usual excellent job in describing procedures which if followed will produce a sound durable patch. Too often patching is a hit and miss proposition and frequently the patch is as inferior as the faulty concrete it replaces.

His observation that "at least equally good bond, and possibly more permanent bond, is obtained when joint surfaces are approaching dryness when the new material is placed on them" points to a condition which may have been partially responsible for faulty patches found too frequently on a recent project. The inspectors zealously required the area, to be patched after faulty concrete had been removed, to be kept wet for at least 12 hr in conduits where forms had been left on for an extended period. The lack of bond occasionally found may have been caused by the surface of the concrete being too wet when the patch was applied. On this project the practice of sounding all shallow patches at regular intervals was adopted. This practice revealed hollow sounding patches after several months which had previously been sound.

One method of insuring bond not mentioned by Mr. Tuthill is to coat the concrete with epoxy resin before placing the patch. Good bond is assured if the necessary precautions are observed.

AUTHOR'S CLOSURE

The additional information on this subject contributed by Messrs. Hawkins, Smith, and Glassgold in their discussions is appreciated. Excellence and serviceability of repairs are so dependent on handcraft operations that it is well to have as many as possible of the necessities and helpful items to this end included.

As it is not our purpose here to debate the virtues and limitations of shotcrete, it is merely suggested that before its use is decided in any case, a careful examination should be made if possible of similar type repairs after several years of exposure in a similar service environment. It is predicted that among what is seen, if it is representative, there will be ample evidence to substantiate the precautions mentioned from the work of ACI Committee 805.

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Discussion of a paper by Earl J. Felt:

Repair of Concrete Pavement*

By H. E. LINDSEY†

The paper by Mr. Felt summarizes the history of bonded patching of concrete and in addition includes important research showing a variety of methods used for preparing the surface prior to patching and the relative effect on bond. This paper is an important contribution to concrete technology.

In the summer of 1957 it was found that some recently completed apron construction at Seymour-Johnson Air Force Base showed spalled joints in addition to short laminations parallel to the surface, but an inch or so below the top of the concrete pavement and adjacent to a few joints. These defects were due to finishing difficulties at the time of placing. The construction techniques outlined in Mr. Felt's paper had been used on several airfields at that time with good success; accordingly, a decision was made to use the thin bonded method to bring the paved area to a uniformly satisfactory condition.

The laminated areas were sounded by hammer and marked. Then the concrete along the joint was removed to a 2 in. depth, as shown in Fig. 11. One area, approximately 12 x 40 ft, was patched to improve a drainage condition. The application of acid was approximately 1 gal. per 50 sq ft, applied to a wetted surface by pumping commercial grade full strength hydrochloric acid through a plastic hose which was coiled at one end and drilled with 1/16-in. diameter holes. This provided a distributor head about 1 ft square. For narrow areas the head could be tilted to prevent acid from spilling on areas that were not to be treated. After foaming of the acid was finished the residue was removed by means of repeated flushing with a high velocity water jet followed by vigorous brooming. Checks were made with litmus paper until a clean surface showing a neutral reaction was obtained.

Since the patching was done during summer months with the danger of developing an air film between the old and new concrete, the surface was lightly dampened with water and a compressed air jet used to prevent any accumulation of moisture in depressions. In all cases the surface of the old concrete was damp and drying and thoroughly cleaned

*ACI JOURNAL, V. 32, No. 2, Aug. 1960 (*Proceedings* V. 57), p. 139. Disc. 57-7 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 9, Mar. 1961 (*Proceedings* V. 57).

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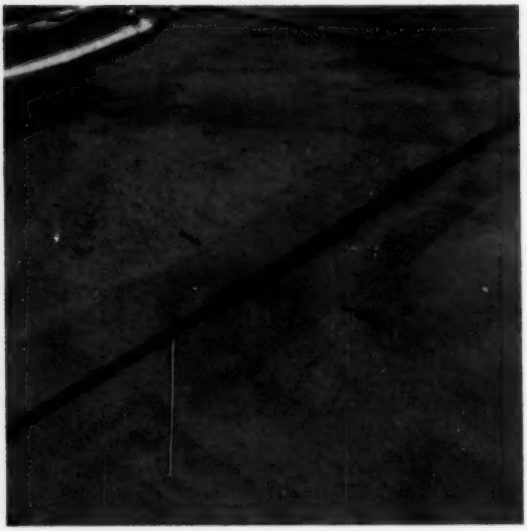

Fig. A — Patched joint placed in 1957 is in perfect condition 3 years later

before grout was applied. It is suggested that the brooms used for spreading acid be confined to that operation and not be used to spread the 1:1 grout.

The value of a spud vibrator in making patches of this type should be emphasized. The vibrator not only densifies the concrete, but by allowing it to rest on the old concrete the entire slab functions as a vibrating table resulting in complete vibration and excellent contact between new and old concrete. After finishing was completed and before the preliminary burlap cure was applied, the edges of the patch were again hand floated to reduce to a minimum any possible shrinkage.

These patches have been most satisfactory (Fig. A). The apron is in constant use by jet planes. The only condition noted that has developed in the last 3 years is a very faint hairline opening along the sawed edge of several of the patches. This has caused no difficulty. A recent survey shows that the patches are all well bonded to the underlying slab.

Discussion of a paper by Bailey Tremper:

Repair of Damaged Concrete with Epoxy Resins*

By M. LEVY and AUTHOR

By M. LEVY†

In the United Kingdom polyester and epoxy resins have been used for a few years as agents for the bonding and repair of precast concrete products. The paper presented by Mr. Tremper has therefore provoked quite a few thoughts and the writer would be grateful for qualification of the following aspects.

Although the title of the report refers to damaged concrete, the context is not quite clear whether repairs are recommended for concrete of poor quality that has deteriorated through weathering. It would be appreciated if the author could qualify the exact recommendations for the application of epoxy resins.

It is perhaps unfair to conclude that the bond strength of the resin is greater than that of the concrete itself from tests on joined broken faces of prisms. These specimens have already been tested in flexure and therefore have incipient weaknesses over the tensile section which would yield on re-test. Perhaps a better method of test to adopt, which we find arrives at the same answer, is to break a beam under two-point loading, then to turn the unit round and join the butt faces together after cleaning and wire-brushing them thoroughly. This beam may then be re-tested over a section of sound concrete when the resin has hardened. Fig. 1 appears to illustrate this method of test but this is not fully qualified in the text.

The illustration of the repair of the large crack by the insertion of pressure fittings can only be applied to cracks of rather large size. Our Research Committee is usually asked to advise on the repair of cracks between 0.015 and 0.100 in. wide where drilling holes to take pressure fittings would only result in filling the crack with dust. In this case we would recommend that the beam be laid on its side, end-supported, and jacked up to ensure trueness and the bottom of the crack pointed up with plasticine. Neat resin would be brushed on at the top until it overflowed and then the plasticine would be pointed up in lifts of 1 in. at a time and the cycle repeated until the section

*ACI JOURNAL, V. 32, No. 2, Aug. 1960 (*Proceedings* V. 57), p. 173. Disc. 57-9 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 9, Mar. 1961 (*Proceedings* V. 57).

†Research Committee for the Cast Stone and Cast Concrete Products Industry, Wexham Springs, Stock Poges, Buckinghamshire, England.

was full of resin. When hard the plasticine would be raked out and the crack pointed up with an extended resin mortar to match the concrete in color—in effect an invisible repair. Units which have been repaired in this way always crack somewhere else when loaded again. It is admitted that this method of repair cannot be applied to fine cracks which may be present in in-situ or erected concrete structures if these cracks are in a horizontal plane. If the cracks are in a vertical plane then the recommendation would be the same as for an individual unit.

Adverting to twin-pack and triple-pack systems, a practice sometimes used in this country which has been found extremely handy is to sell the filler, catalyst, and plasticizing agent in one tin and the epoxy resin in another tin. The catalyst/filler ratio is so proportioned that when the two materials are mixed together to form a mortar-like consistency paste, the correct catalyst/epoxy content is obtained.

Cannot new concrete be bonded quite well to old concrete providing that unsound material, dust, oil, etc. are first removed and a mechanical key achieved by hacking? In addition there is quite an improvement in bond by the application of a polyvinyl acetate stippled key coat to the old concrete which is allowed to harden before topping over with new concrete. It would be a good thing if some laboratory test results had been included in this particular section as we are of the feeling that epoxy cement bonding coats would give too high a strength for the application under consideration.

The author has mentioned an important application of epoxy resin and that is for nonskid toppings, and it would be interesting to know what further applications are being considered in the United States. Does not the author feel that the medium and high viscosity resins are more useful, as they form thicker films and therefore can bond any larger size aggregate of nonskid properties?

Research in the United Kingdom has been mainly confined to the polyester resins rather than the epoxies, not because they are cheaper, c.f. $0.42 per lb to $1.70 per lb, but the following advantages apply.

1. There is not the same dermatitic hazard as curing agents are peroxide and not amide based.

2. Certain types of polyester resin can actually be applied to damp concrete not necessarily surface dry.

3. Polyester resins are not so sensitive to the proportion of catalyst used and some latitude is permissible.

4. A greater range of viscosities is possible and they can be manufactured of a much lower viscosity than the epoxies.

5. It has been found that the polyester resins cured with benzoyl peroxide and accelerated with polysulfides can be made to harden in 5-10 min, and other types of resins catalysed with M.E.K. peroxide and accelerated with cobalt naphthenate hardened in 20 min to 2 hr, so that a complete range of hardening times is achieved.

Polyester twin-packs have been used in this country for post-tensioning operations such as driving post-tensioned piles in series as well as the repair of spalled surfaces of load-bearing units.

AUTHOR'S CLOSURE

Dr. Levy's discussion is much appreciated and it adds to the information available on repairing concrete.

The author has not had personal experience in applications to concrete that has deteriorated seriously through weathering. For this reason the word "damaged" rather than "deteriorated" was purposely used in the title. Reference 1 in the paper indicates apparently successful results if the test described therein is used to assure that the substrate is of adequate soundness.

With respect to injection of adhesive into fine cracks under pressure, the method has been used in cracks in pavements that certainly were less than 0.1 in. in surface width. Cores cut subsequently have shown penetration to full depth of the slab and good bond.

In addition to the demonstrations of bond strength of epoxy adhesives cited in the paper, shear tests of cemented joints between blocks of new concrete have invariably resulted in failure within the concrete itself. Similar results have been obtained with steel bonded to concrete.

The author's experience with polyvinyl acetates is that they provide adequate bond so long as the concrete remains dry, but when exposed to moisture, they soften and lose strength.

When greater film thickness is needed for the embedment of aggregates for nonskid purposes, the desired viscosity of the epoxy adhesive can be obtained more economically by incorporating fine inert filler in low viscosity epoxy than by the use of higher viscosity epoxies. In California experience, the filled epoxies possess adequate bond strength.

Dr. Levy's comments on the use of polyesters are enlightening. Some of the advantages cited can be obtained by varying the composition of the epoxy-curing agent system. Polyesters have been used in California in repairing small spalls in old concrete pavements. Those varieties of polyester with which the author is familiar possess one serious disadvantage, however, in that they tend to saponify in the presence of alkaline solutions such as are developed in fresh concrete. They do not appear to be suitable for bonding fresh concrete to old.

Discussion of a paper by O. N. Kulberg:

Pneumatically Applied Mortar for Restoring Concrete Structures*

By R. W. SPENCER, I. L. TYLER, and AUTHOR

By R. W. SPENCER†

The paper by Mr. Kulberg is of special interest because it gives some long time results of concrete repair work. Many thousands of concrete repairs are made each year and reports are written about a good many within a year of their completion. It is exceptional when anyone reports on a repair over 20 years old. Such a report is especially valuable where both successes and complete or partial failures are considered.

In this connection, it should be pointed out that an application of pneumatically placed mortar will nearly always be unsatisfactory when it is used as a thin shell to cover a joint which shows active movement from day to day or month to month. In other words, it does little good to cover a joint in a dam or heavy concrete structure with a layer of mortar if it is subject to temperature or stress changes which open and close the joint more than one or two hundredths of an inch. The remedy in such a case, of course, is to construct a regular expansion joint at the point of movement.

By I. L. TYLER‡

Mr. Kulberg's paper very ably points out the favorable properties and the limitations of shotcrete, gunite, or pneumatically applied mortar —whichever term you prefer. Mr. Kulberg's opening paragraphs regarding the durability of properly proportioned pneumatically applied mortar can be corroborated by many of us concerned with the useful life of concrete structures. Seldom is durability of the mortar a factor in the performance of shotcreted structures.

But the mortar does shrink excessively, enough to give the appearance of disintegration in some cases of unsightly pattern cracking even though

*ACI JOURNAL, V. 32, No. 2, Aug. 1960 (*Proceedings* V. 57), p. 183. Disc. 57-10 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 9, Mar. 1961 (*Proceedings* V. 57).

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‡Member American Concrete Institute, Research Counselor, Portland Cement Association Research and Development Laboratories, Skokie, Ill.

the individual portions between cracks may be perfectly sound. Frequent dowelling to the original concrete and reinforcement in the mortar is necessary, as Mr. Kulberg points out, and the old concrete must be free from deterioration to assure continuity in the structure. Pattern crazing or cracking is usually more prevalent than with conventional concrete. While somewhat more pattern crazing or cracking is likely to result than might be expected for conventional concrete, the writer agrees with Mr. Kulberg that use of pneumatically applied mortar is an entirely satisfactory means for repairing many old structures.

AUTHOR'S CLOSURE

Mr. Tyler and Mr. Spencer confirmed the author's experience regarding the durability of pneumatically placed mortar as a material and both point out its vulnerability to shrinkage or pattern cracking.

The use of pneumatically placed mortar as a means of restoring deteriorated concrete structures is purely an economic problem and should be recognized as such. It was the author's desire to point out that this economic means of restoration can be successfully used in spite of its shrinkage characteristics when reasonable maintenance precautions are observed.

Sealing shrinkage cracks in pneumatically placed mortar to perpetuate its useful life presents less of a problem than painting steel or any other material to perpetuate its useful life.

Discussion of a paper by Sidney A. Guralnick:

High-Strength Deformed Steel Bars for Concrete Reinforcement*

By FRANK W. CHAPPELL, INGE LYSE, PAUL F. RICE, and WALTER S. SCOTT

By **FRANK W. CHAPPELL**†

More economically constructed reinforced concrete structures may be obtained by the use of one, two, or all of the following materials:

(a) High-strength steel

(b) High-strength concrete

(c) Lightweight concrete

The author confines his report to the first of these. However, all of these materials have been used in a 265 ft high building in so far as limiting codes allow. Concrete weighing 100 lb per cu ft and having a compressive strength of 6000 psi was combined in column design with alloy steel having a yield point of 82,365 psi.‡ The building has been occupied for 4 years without visible structural defect.

The subject report is valuable in the following respects:

1. It supports the ultimate strength method of design (ACI 318-56, Appendix) without which no advantage is gained, under present codes, through the use of high-strength steel in members subject to flexure.

2. It points out the ultraconservatism of limiting reinforcement to an ultimate stress of 60,000 psi, when stronger steel (ASTM A 431) is readily available. Alloy steel bars, more ductile than A 431, easily weldable and having yield points up to 90,000 psi, can be had on mill order.

3. It furnishes test data on which an amendment to the ACI Building Code may be based. Paragraph 306 should make provision for suitable working stresses for the new steels, ASTM A 431 and A 432.

4. It gives a logical method of predicting deflections of reinforced concrete beams, whether simply supported or restrained, rectangular or T-setions.

5. It provides valuable data for predicting widths of cracks. Permissible cracking is currently under investigation by ACI Committee 339, Allowable Stresses in Reinforcement, and was not treated in the subject.

*ACI JOURNAL, V. 32, No. 3, Sept. 1960 (*Proceedings* V. 57), p. 241, Disc. 57-12 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 9, Mar. 1961 (*Proceedings* V. 57).

†Member American Concrete Institute, Consulting Engineer, Dallas, Tex.

‡Chappell, Frank W., "High Strength Steel and Concrete Result in Minimum Column Sizes," ACI JOURNAL, V. 29, No. 11, May 1958 (*Proceedings* V. 54), pp. 929-938.

The writer found the report interesting and informative. If a companion research project could be undertaken, using lightweight concrete (100 lb per cu ft) having clyinder strengths up to and including 6000 psi, three approaches would be opened to more efficiently designed structures.

By INGE LYSE*

The author presents some interesting results from tests of concrete beams reinforced with high-strength deformed bars. In connection with these results it may be of interest to know that in the proposal of a new Norwegian building code for reinforced concrete the maximum stress permitted in the reinforcement is as high as 3100 kg per sq cm (44,000 psi) for deformed bars of less than 16 mm (5/8 in.) diameter with minimum yield-point strength of 6000 kg per sq cm (85,000 psi). But in addition it is required that the maximum width of crack computed by a given formula must not exceed certain maximum values depending on the condition of exposure of the structure.

Regarding the method of design, the classical (elastic) method and the ultimate strength method are given equal standing with the definite requirement that both shall give the same results.

The carrying capacity of a reinforced concrete beam is independent of the method used for its design, and it seemed entirely unjustified to allow a lower factor of safety with one method of design in preference to another.

The author finds that the formula for determining the modulus of elasticity of concrete presented by the writer† and based on tests at Fritz Engineering Laboratory, Lehigh University, is still useful. During recent years working with Norwegian cements and aggregates, the writer has found that the original formula:

$$E_c' = 1.8 \times 10^6 + 460 f_c'$$

should be changed to

$$E_c' = 1.42 \times 10^6 + 500 f_c''$$

or in the metric system

$$E_c' = 100{,}000 + 500 f_c''$$

where f_c'' is the cube strength of the concrete. It is rather surprising that the original formula was found valuable for computing deflections at the University of Illinois and at Cornell University, since both universities used aggregates vastly different from those used at Lehigh University in the 1930's.

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†Sutherland, Hale, and Reese, Raymond C., *Introduction to Reinforced Concrete Design*, John Wiley and Sons, Inc., New York, 1943, p. 36.

By PAUL F. RICE*

The author is to be congratulated for providing much needed quantitative answers applicable to the use of materials common in United States design practice. The comparisons with procedures given in the ACI Building Code are especially valuable in evaluating the test results. Such comparisons provide a practicing engineer with the necessary confidence and with any special precautions necessary in using the Code. Especial care, however, is required in discussion of test results and in drawing conclusions related to use of the code. Such discussions and conclusions should not only be technically correct for study by other research workers, but also should avoid misleading casual readers which includes most practicing engineers.

In particular, some further amplification would have been desirable in stating the conclusion that "an ultimate strength theory which is based on the assumption that at failure the stress in the compression reinforcement is at the yield point, irrespective of distance of the compression steel center-of-gravity from neutral axis, is not tenable." The statement is obvious but not particularly helpful and could be actually misleading.

Our present code for ultimate strength design (USD) does not require that all compressive reinforcement, regardless of distance from the neutral axis, be assumed to be at yield point. The basic assumption, Section A603(c), is merely that the stress at ultimate load shall not be assumed *greater* than the yield point.

Since over-reinforced flexural members will seldom be required in (USD) practice, and since compression reinforcement in the top flange of a T-beam is rarely, if ever, economical, even in working stress design (WSD), the behavior of such reinforcement is of little practical interest. Furthermore, the specimens tested contained equal amounts of tension and compression reinforcement which is outside the applicable limits of the present code provision on compressive reinforcement, Eq. (A3). The top portion of the author's Fig. 2 shows that such reinforcement does develop a small compressive stress until the diagonal tension crack intersects it when, as would be expected, it develops tension.

Fig. 2, however, is typical of beams failing prematurely due to diagonal tension, not the usual design. The top portion of Fig. 5 shows zero strain in compression reinforcement in the flange up to flexural failure, indicating that the steel was located right on the neutral axis, again not the usual design practice.

The section on ultimate strength design of T-beams in the present code (Section A607) makes no provision for compression reinforcement

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indicating that its use in the flange of a T-beam is not even contemplated.

At a support, where the normal upright T-beam behaves like a rectangular beam with a width that of the stem, use of compression steel in the stem is common under WSD practice. This condition is covered by Section A606 which provides that the ultimate moment *shall not exceed* that developed if the compression steel is assumed to be at yield. The lower portions of Fig. 2 and 5 show strain in compression steel in the stem over a support. In both cases, a compressive stress compatible with the distance from the neutral axis developed. Fig. 2 shows that this stress may reverse to tension if the neutral axis moves downward as diagonal tension cracks cross the compressive steel. Fig. 5 shows that, with stirrups to prevent diagonal cracking, the compression steel carries compression increasing with load up to approximately 30,000 psi. Presumably, creep under long-continued load should increase this stress, perhaps doubling it.

For the author's purposes since the failures in flexure occurred under positive moment, it was quite proper to neglect the compression steel in calculating flexural strength. The statement that this neglect is justified because "little if any stress was being transferred to the compression steel" could well be qualified by adding "in the flange."

The combined effect of omitting such a qualification and the conclusion as stated can be misleading to any casual reader of the paper. The conclusion could have been made much more helpful to designers by turning it around, dropping the implication that compression steel is necessarily ineffective, and providing a positive rather than negative conclusion for its effective use.

Perhaps the basic assumption concerning steel stress in our present code, A603(e), could have been more completely stated somewhat as follows:

> "For strains greater than that corresponding to the yield point stress, the stress in reinforcement shall be assumed equal to the yield point stress. For strains less than that corresponding to the yield point stress, the stress in the reinforcement shall be assumed as 30,000,000 psi times the strain."

In Section A606, concerning compressive reinforcement, perhaps it should have been noted that in practical design of most doubly reinforced beams the compression steel is close to the face of maximum concrete compression and that the compression steel stress can be assumed at yield point. It could also be noted that Eq. (A3) applies only when compression reinforcement is at yield point, but that strain should be checked at the compression steel to confirm that yield has occurred. The basic assumption for use in such checking is given in Section A603(a), but perhaps some of the equations derived from this assumption could also have been provided in the code.

Obviously, the code provisions on USD could be improved. In addition, no research result under discussion is incompatible with the code as it stands. The natural inferences that might be drawn from a quick reading of the paper would be unfortunate if the impressions left are that compression steel is necessarily ineffective and that the present code is unsafe.

By WALTER S. SCOTT*

I wish to comment briefly on the characteristics of the steel used in this experiment. This paper has contributed valuable information for use in connection with high-strength reinforcing bars.

One feature brought out is the type of stress-strain curve obtained for hot rolled alloy steel bars. The author makes the statement that "Alloy steels can be produced which have stress-strain curves with sharp, definite yield points . . .". We question whether this is true for most alloy grades of bar steel. It is assumed that reinforcing bars in the as-rolled condition is referred to, as this is the type of reinforcing bar usually used.

As there has been considerable discussion in engineering circles as to the results from using bars with a gradually yielding type of steel, as against the sharp yielding type, it is important to point out that the

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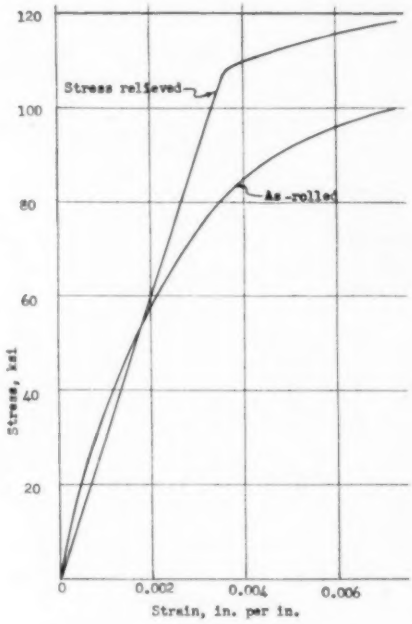

Fig. A—Stress strain curve for Grade 4145

Composition

C	Mn	P	S	Si	Ni	Cr	Mo
0.46	0.83	0.015	0.024	0.23	1.10	0.91	0.19

Size, in.	Yield strength, psi	Tensile strength, psi	Elongation, percent	Treatment
1/2	90,500	191,600	4.5	Hot-rolled
1/2	114,200	139,000	9.5	Stress relieved

grades of alloy steel reinforcing bars most commonly furnished to ASTM A 431 have the gradual yielding type of stress-strain curve.

Fig. A through F are typical stress-strain curves representing alloy steel bars of the grades commonly used for reinforcing furnished to meet ASTM A 431 with 75,000 psi minimum yield point. It can be seen from the stress-strain curve representing the popular 4100 grades, which is also typical of the majority of alloy steels in the as-rolled condition, that the yield occurs gradually. Additional curves illustrating this condition are furnished for Grades 8642 and 4063.

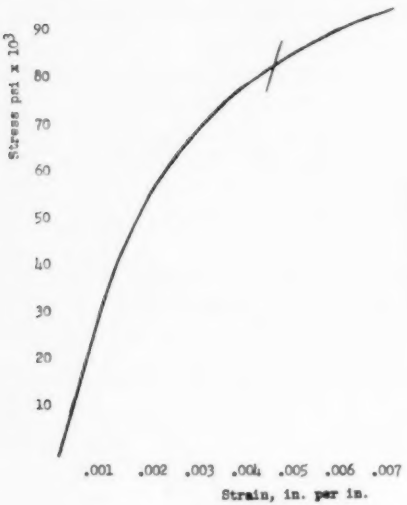

Fig. B—Stress-strain curve for Grade 4140

Deformed bar size #14
Tested at 0.505 section as-rolled
Yield strength 0.2 percent offset 84,210 psi
Ultimate strength 123,800 psi
Elongation 17 percent, Gage 2 in.
Reduction of area 46.7

Composition

C	Mn	P	S	Si	Cu	Ni	Cr	Mo
0.37	0.79	0.010	0.028	0.27	—	—	0.81	0.19

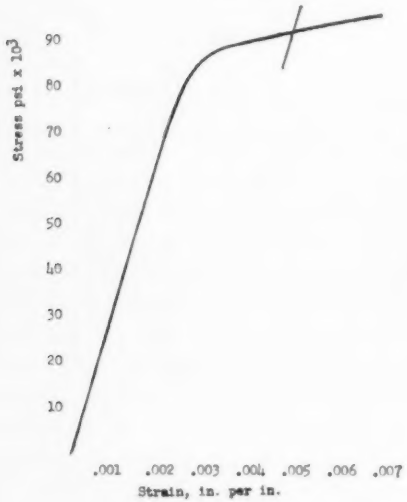

Fig. C—Stress-strain curve for Grade 9260

Deformed bar size #7
Tested at full section as-rolled
Yield strength 0.2 percent offset 92,800 psi
Ultimate strength 148,500 psi
Elongation 4.5 percent, Gage 8 in.
Reduction of area —

Composition

C	Mn	P	S	Si	Cu	Ni	Cr	Mo
0.62	1.02	0.038	0.023	2.10	—	—	—	—

Also included are stress-strain curves for 9260 and carbon steel 1065. It can be seen that the 9260 curve (similar to one of the grades used in this experiment) is relatively sharp at the yield point and is one of the few alloy steels that have such a sharp yielding curve. For comparison a curve is included representing carbon steel of 1065 grade. This is a true sharp yield stress-strain curve.

Another interesting fact is that alloy steels with the gradually yielding type of stress-strain curve can be changed to the relatively sharp yielding type by giving them a stress relief type of heat treatment. Included is a typical curve illustrating the stress-strain curve before and after the stress relief heat treatment.

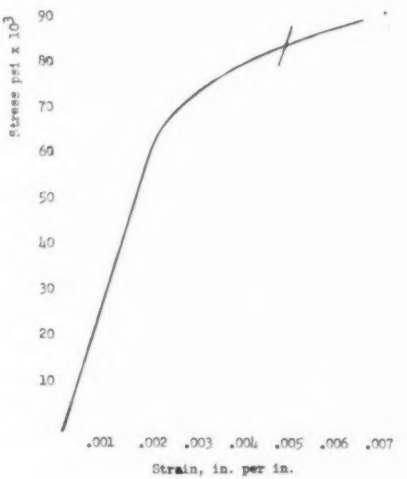

Fig. D—Stress-strain curve for Grade 4063

Deformed bar size #6
Tested at full section as-rolled
Yield strength 0.2 percent offset 85,800 psi
Ultimate strength 136,000 psi
Elongation 10.0 percent, Gage 8 in.
Reduction of area —

Composition

C	Mn	P	S	Si	Cu	Ni	Cr	Mo
0.62	0.90	0.016	0.021	0.30	0.12	0.16	0.13	0.23

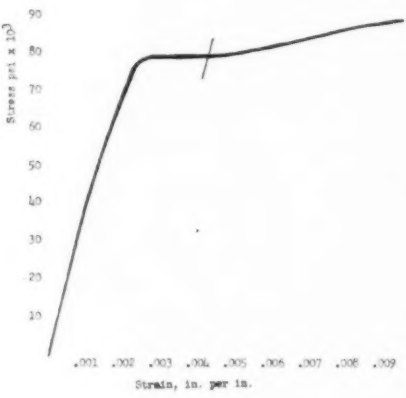

Fig. E—Stress-strain curve for Grade 1065

Deformed bar size #4
Tested at full section as-rolled
Yield strength 0.2 percent offset 79,420 psi
Ultimate strength 137,400 psi
Elongation 14.5 percent Gage 2 in.
Reduction of area —

Composition

C	Mn	P	S	Si	Cu	Ni	Cr	Mo
0.66	0.84	0.037	0.035	0.25	0.03	0.03	0.02	0.01

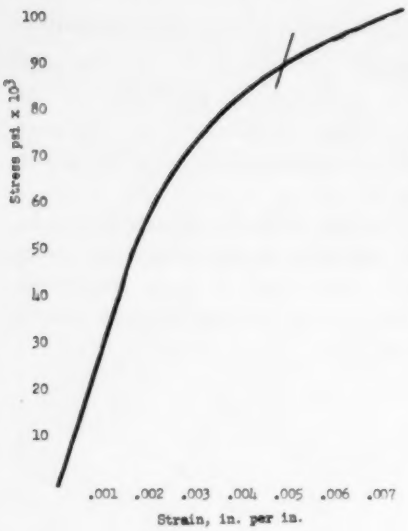

Fig. F—Stress-strain curve for Grade 8642

Deformed bar size #6
Tested at full section as-rolled
Yield strength 0.2 percent offset 91,500 psi
Ultimate strength 135,000 psi
Elongation 8.5 percent, Gage 8 in.
Reduction of area —

Composition

C	Mn	P	S	Si	Cu	Ni	Cr	Mo
0.45	0.93	0.018	0.013	0.33	0.13	0.55	0.55	0.19

Use of 75,000 psi yield reinforcing bars has increased, and would be still greater if sufficient data were available to warrant ACI Committee 318, Standard Building Code, allowing its use in beams. As the yield strength of the alloy bars used in this experiment are over 75,000 psi it may be worthwhile recalculating the data to obtain some idea of a safe maximum design stress for 75,000 psi yield reinforcing bars in beams. The author's statement that "from the standpoint of flexural strength alone, the 60,000 psi limits on maximum steel stress is overly conservative," would indicate that 75,000 psi yield strength bars should be considered for use in beams.

Discussion of a paper by Stanton Walker and Delmar L. Bloem:

Effects of Aggregate Size on Properties of Concrete*

By K. M. ALEXANDER and J. WARDLAW, IQBAL ALI and A. B. LINGAM, RICHARD J. FRAZIER, MARVIN J. HAWKINS, J. HOLLEB, E. L. HOWARD, M. F. MACNAUGHTON, A. R. MEAD, HARRY H. MITCHELL, M. SPINDEL, SVEN THAULOW, BAILEY TREMPER, LEWIS H. TUTHILL, PAUL S. WRIGHT, N. G. ZOLDNERS, and AUTHORS

By K. M. ALEXANDER and J. WARDLAW†

The results which the authors published in their paper are most interesting and we would like to add evidence, from our recent work[22] on cement-aggregate bond, which both supports the conclusions drawn by Messrs. Walker and Bloem, and offers a possible explanation for some of the behavior they observed.

Using a method similar to that described in a previous issue of this JOURNAL,[23] we have measured the strength of the bond between hydrated portland cement paste and the surface of sawn aggregate cubes, ranging in size from ½-in. to 3-in. The results show that cement-aggregate bond strength increases rapidly with decreasing size of aggregate. For example, at a water-cement ratio of 0.35 by weight, and at the age of 7 days, the strength of the bond between an ordinary portland cement and basalt aggregate (Fig. A) rose from 100 psi for 3-in. aggregate, to 930 psi for ½-in. aggregate. These experiments are being extended to include aggregate smaller than ½-in.

This decrease in cement-aggregate bond strength with increasing size of aggregate offers a likely explanation for the reduction in the strength of concrete, at a given water-cement ratio, which Messrs. Walker and Bloem observed when they increased the maximum size of the coarse aggregate.

It appears that the same trends noted when the maximum size of aggregate used by Messrs. Walker and Bloem was reduced to ⅜-in., continue when an even lower size limit is set. For example, if the maximum size of aggregate is restricted to 3/16-in., the mix becomes a

*ACI JOURNAL, V. 32, No. 3, Sept. 1960 (*Proceedings* V. 57), p. 283. Disc. 57-13 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 9, Mar. 1961 (*Proceedings* V. 57).

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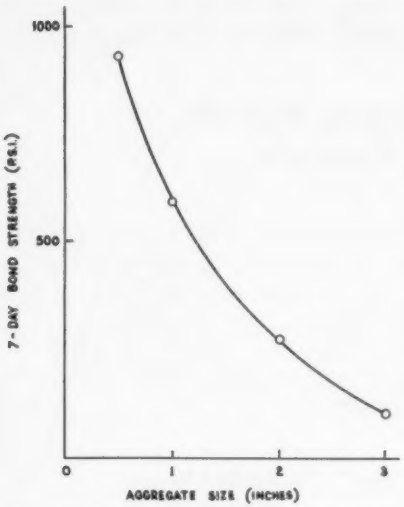

Fig. A—Dependence of cement-aggregate bond strength on size of aggregate. Data are for basalt aggregate bonded to ordinary portland cement paste with a water-cement ratio of 0.35 by weight

mortar and, according to Jones and Kaplan,[24] the flexural strength of mortar is greater than that of the corresponding concrete.

In the earlier paper,[5] which Messrs. Walker and Bloem presented before the Highway Research Board, and to which they refer in their present publication, the authors state that the reproducibility of strength tests was poorest for concrete containing the largest maximum size of aggregate. This behavior too can be attributed to the dependence of cement-aggregate bond strength on size of aggregate. For example, with the specimens used for determining flexural strength, zones of particular weakness exist at the interface between the matrix and the largest pebbles. For these weak links to have their greatest effect on strength, one of the largest pebbles must be present in that region of the beam which is in maximum tension and in addition, with irregular shaped pebbles, a weakly bonded surface must be suitably orientated. Taking the extreme example of beams containing the largest maximum size of aggregate, these conditions of pebble location and interface orientation are particularly hard to fulfill because the really large pebbles in such beams are few in number. There is thus ample scope for the relatively large variations in strength noted in this particular section of the project.

A possible reason for the bond strength-size effect is that in cement paste the process of hardening may be accompanied by dimensional changes which are large enough to give rise to considerable internal stress in the paste film in contact with the aggregate surface, when this tendency of the paste to move is opposed by the aggregate. The larger the surface bonded, the greater the stress developed and the smaller the apparent bond strength. A rather similar situation applies in the case of the restraint which different sizes of aggregate exert on portland cement paste during drying shrinkage.[25]

It is considered that the bond strength-size effect in concrete will probably not be as marked as that shown by isolated cement-aggregate

interfaces studied here, because, in concrete, the largest aggregate pebbles are not embedded in neat cement paste, but in a matrix, the dimensional changes of which are reduced by the stabilizing influence of the smaller pebbles and sand grains which it contains. Conversely, an increase in paste content will tend to increase any effect which the bond strength-aggregate size relationship may have on the strength of the concrete. This may be the reason why large sizes of aggregate caused a more pronounced decrease in strength in the concretes of higher cement factor, as reported by Messrs. Walker and Bloem. Similarly, higher paste contents in the lean air-entrained concretes than in the non-air-entrained concretes could explain why the leanest air-entrained mixes tended to have a greater strength reduction with increasing aggregate size than did the corresponding non-air-entrained mixes.

The question of the effect of maximum aggregate size on concrete strength at constant water-cement ratio has a direct bearing on the problem of whether mortar and coarse aggregate should be regarded as the two phases of which concrete consists, or whether the two phases to be considered are paste and all aggregate, regardless of size. However, the fact that bond strength decreases with increasing size of aggregate suggests that neither view is correct, and that plain concrete should be regarded as a material in which there are specific zones of weakness, the most important of which are located at the junction between the matrix and the largest aggregate pebbles. There are of course many reasons why even this hypothesis should not be taken as a hard and fast rule. For example, the largest aggregate pebbles might have a mineralogical composition or surface texture which could lead to the formation of an exceptionally strong bond[23] or, since cement-aggregate bond strength, relative to that of the paste, increases with age,[23] the strength of the bond to even the largest pebble may, in time, exceed that of the adjacent matrix.

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By IQBAL ALI* and A. B. LINGAM†

The data presented by the authors regarding the effect of maximum aggregate size on the strength of concrete is extremely interesting and appears to lend considerable support to the traditional practice of limiting the maximum size of aggregate to ¾ in. or 1 in. for structural concrete, even when much larger sizes are permissible otherwise. Significant differences in concrete strength for the same water-cement ratio, when the type, size, or grading of aggregate is varied, have been reported by several investigators. Due to the absence of a rational explanation for such differences, however, the water-cement ratio continues to remain the only commonly accepted basis for estimation of concrete strength.

The writers would like to take this opportunity to suggest a possible explanation for the observed reduction in the concrete strength, for the same water-cement ratio, when the maximum size of aggregate used is either smaller or larger than a certain critical range. The concept presented is meant to be viewed as a tentative hypothesis only, as the writers are not yet in a position to fully substantiate the same quantitatively.

If fracture of the aggregate particles is precluded, failure of an aggregate-binder system can take place either entirely through the binder or partially through the binder and partly along the binder-aggregate interface depending on the mode which corresponds to the least over-all resistance. The strength of the system may thus be expected to depend on the strength of the binder, the bond strength between the binder and aggregate surface, and the relative proportion of the effective area of the failure surface passing through the binder and along the binder-aggregate interface.

The water-cement ratio would effect the strength of the cement paste, and depending on the nature of the aggregate surface, the bond strength also, but not necessary in the same manner or to the same extent. The relative contribution of these two modes of resistance towards the over-all strength would, on the other hand, be determined by the geometry of the system.

It might therefore be argued that an increase in the maximum size of aggregate would tend to decrease the strength of concrete as the resulting decrease in voids would permit a shorter path of rupture through the cement paste and the reduced specific surface would shorten the path along the aggregate-binder interface. As the binder strength and bond resistance may, as mentioned earlier, vary at different

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rates with change in water-cement ratio, the extent to which the concrete strength is reduced when larger aggregates are used may not be the same for all water-cement ratio levels. Use of smaller aggregate sizes would in a like manner be expected to result in a higher strength at a given water-cement ratio. A counteracting factor has, however, to be considered in this case. The volume of incidentally entrapped air tends to increase rapidly with the specific surface of the aggregate. Reduction in maximum aggregate size can thus result in increased strength only to a certain extent beyond which the weakening effect of the air would predominate. The writers have had occasion to study the strength of concrete and mortar mixes made with very fine sands when it was found that a simple allowance for the air content, by considering it as an equal volume of water, was all that was necessary to explain the lower than normal strengths at given water-cement ratio levels.[26] Data reported by other workers[27-29] analyzed in this context also seemed to support the above conclusion.

The general concept outlined above, indicates the possibility of expressing the strength of concrete in terms of the following variables:

(1) Strength of the cement paste alone

(2) Bond strength between the cement paste and aggregate surface

(3) Geometry of the aggregate-binder system

It should be possible to express the first two as individual functions of the water-cement ratio (including if necessary, a correction for air content). In addition a coefficient representing the surface characteristics of the aggregate would also be involved in the second. The geometry of the system, which would determine the relative contribution of the two modes of resistance to the effective strength, would perhaps involve the relative volumes of aggregate and binder and a parameter representing the area of the aggregate-binder interface available.

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By RICHARD J. FRAZIER*

The paper "Effects of Aggregate Size on Properties of Concrete" by Messrs. Walker and Bloem, as well as an earlier paper presented before the Highway Research Board,[5] reports results that conflict with long established concepts. Since the advent of the water-cement ratio theory in 1917, it has been a standard industry practice to proportion all mixes with the lowest practical water content, with the view of achieving maximum strength for a given cement content. In view of the fact that the larger the aggregate that can be used in concrete, the less water required, it was to be expected, based on the water-cement ratio theory, that the larger the aggregate used, the greater the strength of the concrete.

When the NRMCA reports appeared indicating that no appreciable gain in strength was obtained by using aggregates larger than ¾ in., Robert F. Porter, vice-president of Harry T. Campbell Sons' Corp., felt that these conflicting results should be checked out by a series of tests using their own aggregates. Accordingly, in June, 1959, a limited program was devised to study the effect of maximum size of coarse aggregate on concrete strengths for plain, air-entrained, and retarded concrete mixes. In April, 1960, a supplemental investigation was undertaken to investigate the effect of maximum size of coarse aggregate on strength and durability of concrete presently used in pavement mixes of the state of Maryland. This discussion covers the results of our findings.

TEST SERIES A

Batches of concrete using 6, 7, 8 and 9 sacks of cement with gravel coarse aggregate in four maximum sizes (⅜, ⅝, 1, and 2 in.) and crushed limestone coarse aggregate in four maximum sizes (½, ¾, 1, and 2 in.) were produced. For each coarse aggregate size and cement content, plain, air-entrained, and retarder batches were produced. From each batch, four 6 x 12-in. cylinders and two 6 x 24-in. beams were cast for compressive and flexural strength tests, respectively.

Coarse aggregates were selected by obtaining material from routine production. Care was exercised to obtain as nearly representative a sample of daily production as possible, but no attempt was made to screen or otherwise regrade the material. The sand was a well graded White Marsh natural concrete sand and was used as a fine aggregate throughout the test series. Table A gives the gradings and characteristics of coarse and fine aggregates used.

Type I portland cement for these tests was obtained from a single car, a sufficient amount being set aside to take care of all batches. The air-

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entraining agent used was a commercially available product and was proportioned to provide air contents in the mixes in accordance with "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)" but modified for some of the coarse aggregate sizes. To achieve the designed air contents over the extended test period, proportions of the air-entraining agent were varied to meet the changing demands of ambient temperature, changes in sand gradation, etc., and ranged from 0.2 to 0.5 oz per sack of cement. The water reducing-densifying retarder additive was of the calcium-lignosulfonate type. This additive was used at the rate of 4 oz per sack of cement for each mix in the series. Propor-

TABLE A — CHARACTERISTICS OF AGGREGATES AS USED IN CONCRETE, SERIES A

Gravel

Sieve size or item	Coarse aggregate maximum size				Fine aggregate
	⅜ in.	⅝ in.	1 in.	2 in.	
	Grading, cumulative percents passing each size				
2½ in.	—	—	—	100.0	—
2 in.	—	—	—	91.5	—
1½ in.	—	—	100.0	80.5	—
1 in.	—	—	99.1	20.2	—
¾ in.	—	100.0	80.5	1.1	—
½ in.	—	94.2	56.0	—	—
⅜ in.	100.0	68.9	37.0	—	100.0
No. 4	20.8	12.9	3.7	—	97.4
No. 8	2.6	4.2	0.9	—	79.7
No. 16	1.2	—	—	—	65.9
No. 30	0.8	—	—	—	49.5
No. 50	0.4	—	—	—	21.8
No. 100	—	—	—	—	3.6
No. 200	—	—	—	—	—
Fineness modulus	5.74	6.14	6.79	8.18	2.82
Specific gravity bulk SSD	2.63	2.63	2.63	2.63	2.63

Stone

Sieve size or item	Coarse aggregate maximum size				Fine aggregate
	½ in.	¾ in.	1 in.	2 in.	
	Grading, cumulative percents passing each size				
2½ in.	—	—	—	100.0	Same
2 in.	—	—	—	100.0	as
1½ in.	—	—	100.0	99.4	above
1 in.	—	100.0	99.8	44.4	
¾ in.	—	96.8	85.9	10.4	
½ in.	100.0	49.8	34.4	1.7	
⅜ in.	85.0	24.7	11.0	1.5	
No. 4	14.4	3.6	5.0	1.3	
No. 8	3.5	3.0	4.4	—	
No. 16	3.1	—	—	—	
No. 30	2.7	—	—	—	
No. 50	2.0	—	—	—	
No. 100	1.3	—	—	—	
No. 200	—	—	—	—	
Fineness modulus	5.88	6.74	6.71	7.67	
Specific gravity bulk SSD	2.82	2.82	2.82	2.82	

TABLE B — CHARACTERISTICS OF FRESH AND HARDENED CONCRETE, SERIES A: GRAVEL AGGREGATE

Design cement, sacks per cu yd	Maximum size aggregate, in.	Calculated cement, sacks per cu yd	F.M. total aggregate*	b/b₀†	Sand, percent total aggregate	Water, gal. per sack	Percent air	Compressive strength 28 days	Flexural strength 28 days
Group 1—Non-air-entrained concrete									
6	3/8	5.71	4.075	0.490	57.0	7.52	3.0	4350	620
	5/8	5.87	4.520	0.570	48.8	6.34	2.4	5064	723
	1	6.04	5.253	0.670	38.7	5.53	1.5	5756	698
	2	6.02	6.412	0.768	33.0	5.09	1.1	5353	640
7	3/8	6.84	4.107	0.490	55.9	5.70	2.7	6001	742
	5/8	6.88	4.564	0.570	47.5	5.23	2.4	5800	765
	1	7.04	5.317	0.670	37.1	4.83	1.5	5983	763
	2	6.91	6.503	0.768	31.3	4.93	0.9	5919	677
8	3/8	7.79	4.154	0.490	54.3	5.34	2.9	6543	850
	5/8	7.89	4.623	0.570	45.7	4.88	2.0	6012	848
	1	7.98	5.401	0.670	35.0	4.66	1.4	6225	757
	2	7.91	6.621	0.768	29.1	4.34	0.9	5706	640
9	3/8	8.78	4.201	0.490	52.7	4.76	2.6	6343	873
	5/8	8.88	4.686	0.570	43.8	4.63	1.8	6201	909
	1	9.01	5.492	0.670	32.7	4.18	1.3	6573	865
	2	8.89	6.749	0.768	26.7	4.24	0.8	5376	683
Group 2—Air-entrained concrete									
6	3/8	5.94	4.289	0.528	49.7	6.30	8.0	3796	621
	5/8	5.95	4.749	0.606	41.9	5.38	8.0	4236	677
	1	6.12	5.492	0.704	32.7	5.35	4.5	4822	596
	2	5.97	6.614	0.786	29.2	5.14	4.2	4093	567
7	3/8	7.03	4.342	0.528	47.9	4.92	7.5	4554	713
	5/8	7.07	4.818	0.606	39.8	4.79	6.2	5052	736
	1	7.12	5.587	0.704	30.3	4.52	5.3	4922	642
	2	7.02	6.754	0.786	26.6	4.67	4.2	4297	708
8	3/8	8.12	4.399	0.528	45.9	4.83	6.6	5382	804
	5/8	8.08	4.908	0.606	37.1	4.51	6.2	4589	773
	1	8.26	5.718	0.704	27.0	4.40	3.8	5249	765
	2	8.07	6.921	0.786	23.5	4.21	4.0	4474	795
9	3/8	9.15	4.461	0.528	43.8	4.45	6.6	5105	848
	5/8	9.23	4.988	0.606	34.7	4.24	5.0	5205	873
	1	9.21	5.817	0.704	24.5	3.93	4.2	5022	748
	2	8.97	7.066	0.786	20.8	4.06	3.8	4114	659
Group 3—Retarded concrete									
6	3/8	5.86	4.096	0.490	56.3	6.34	6.0	5187	648
	5/8	6.08	4.543	0.570	48.1	5.54	4.1	5886	686
	1	6.01	5.289	0.670	37.8	5.47	3.6	6119	640
	2	5.97	6.464	0.768	32.0	5.01	3.2	5594	621
7	3/8	7.05	4.129	0.490	55.2	5.65	4.5	6537	700
	5/8	7.08	4.590	0.570	46.7	4.66	4.7	6631	738
	1	7.30	5.353	0.670	36.2	4.18	3.1	6826	769
	2	7.00	6.555	0.768	30.3	4.30	3.0	6260	702
8	3/8	8.15	4.175	0.490	53.6	4.82	4.7	7663	939
	5/8	8.18	4.649	0.570	44.9	4.74	3.6	6923	819
	1	8.17	5.440	0.670	34.0	4.83	2.4	6652	657
	2	7.84	6.680	0.768	28.0	3.87	4.0	6520	619
9	3/8	9.07	4.225	0.490	51.9	4.33	4.5	7392	927
	5/8	9.22	4.716	0.570	42.9	4.22	3.5	7987	857
	1	9.28	5.535	0.670	31.6	3.93	2.6	7221	767
	2	8.99	6.813	0.768	25.5	4.02	2.0	6233	667

*Fineness modulus of combined fine and coarse aggregates.

†Ratio absolute volume of coarse aggregate per unit volume of concrete to absolute volume of coarse aggregate per unit volume of dry rodded coarse aggregate (equivalent to dry rodded volume of coarse aggregate per unit volume of concrete)

TABLE C — CHARACTERISTICS OF FRESH AND HARDENED CONCRETE, SERIES A: STONE AGGREGATE

Design cement, sacks per cu yd	Maximum size aggregate, in.	Calculated cement, sacks per cu yd	F.M. total aggregate*	b/b_o†	Sand, percent total aggregate	Water, gal. per sack	Percent air	Compressive strength 28 days	Flexural strength 28 days
Group 1—Non-air-entrained concrete									
6	½	5.92	4.356	0.583	49.8	7.26	2.1	4886	755
	¾	6.00	4.976	0.629	45.0	6.57	1.3	5090	753
	1	5.97	5.026	0.651	43.3	6.58	1.6	5842	761
	2	6.00	5.681	0.710	41.0	5.82	1.4	6101	913
7	½	6.87	4.399	0.583	48.4	6.60	2.2	5789	828
	¾	7.10	5.031	0.629	43.6	5.35	1.4	6832	978
	1	6.99	5.084	0.651	41.8	5.30	1.3	7156	930
	2	6.97	5.754	0.710	39.5	5.02	1.5	6137	900
8	½	7.89	4.454	0.583	46.6	5.37	2.0	6284	994
	¾	8.05	5.109	0.629	41.6	5.03	1.5	7109	1098
	1	8.03	5.158	0.651	39.9	4.76	1.3	7421	1031
	2	7.90	5.852	0.710	37.5	4.83	1.4	6302	813
9	½	8.91	4.560	0.583	45.1	4.76	1.9	6596	1050
	¾	9.01	5.172	0.629	40.0	4.34	1.4	8476	1248
	1	8.95	5.224	0.651	38.2	4.36	1.4	7817	1107
	2	8.89	5.957	0.710	35.3	4.38	1.3	7061	861
Group 2—Air-entrained concrete									
6	½	6.17	4.566	0.619	42.9	6.27	6.1	4922	825
	¾	6.18	5.215	0.664	38.9	5.83	5.0	4922	817
	1	6.04	5.263	0.685	37.2	5.96	5.0	5364	786
	2	6.10	5.943	0.742	35.6	5.49	3.6	5064	759
7	½	7.24	4.632	0.619	40.8	5.54	5.5	5517	848
	¾	7.11	5.301	0.664	36.7	5.17	5.5	4987	811
	1	7.20	5.349	0.685	35.0	5.12	4.4	6396	938
	2	7.05	6.050	0.742	33.4	4.97	3.8	4716	744
8	½	8.14	4.702	0.619	38.5	5.03	5.6	5347	863
	¾	8.08	5.395	0.664	34.3	4.89	4.8	5824	1015
	1	8.21	5.442	0.685	32.6	4.54	4.6	6101	975
	2	7.98	6.162	0.742	31.1	4.48	4.1	5394	738
9	½	9.07	4.775	0.619	36.1	4.69	5.7	5152	892
	¾	9.19	5.490	0.664	31.9	4.36	4.9	6903	1115
	1	9.30	5.539	0.685	30.1	4.04	4.1	6849	1107
	2	9.01	6.278	0.742	28.7	4.08	3.9	5476	931
Group 3—Retarded concrete									
6	½	6.07	4.414	0.583	47.9	6.75	4.3	5535	771
	¾	6.19	5.055	0.629	43.0	5.88	3.8	6239	890
	1	6.04	5.088	0.651	41.7	5.80	5.7	6632	796
	2	5.86	5.745	0.710	39.7	5.93	5.8	6178	729
7	½	7.00	4.460	0.583	46.4	5.92	5.4	6390	815
	¾	7.18	5.117	0.629	41.4	5.30	3.2	7050	894
	1	7.00	5.166	0.651	39.7	4.93	6.4	7321	873
	2	7.07	5.817	0.710	38.2	4.72	3.1	6650	761
8	½	8.06	4.518	0.583	44.5	4.87	6.6	6909	925
	¾	8.18	5.199	0.629	39.2	4.50	5.6	8671	1032
	1	8.12	5.252	0.651	37.5	4.18	6.0	8430	1017
	2	8.00	5.924	0.710	36.0	4.29	3.4	6915	873
9	½	9.20	4.570	0.583	42.8	4.42	5.5	7616	1023
	¾	9.11	5.271	0.629	37.5	4.05	5.9	9520	1157
	1	8.87	5.341	0.651	35.2	4.13	6.4	8642	1017
	2	9.03	6.031	0.710	33.8	3.92	2.7	7551	912

*Fineness modulus of combined fine and coarse aggregates.

†Ratio absolute volume of coarse aggregate per unit volume of concrete to absolute volume of coarse aggregate per unit volume of dry rodded coarse aggregate (equivalent to dry rodded volume for coarse aggregate per unit volume of concrete).

tions of fine to coarse aggregate were designed to produce a degree of workability suitable for placement in pavements. All mixes were tested for slump and unit weight. Air content was measured by a pressure meter. All batches were mixed in a 3-cu ft mixer and batch sizes were standardized at 7.3 percent of a cu yd. Mixing was adjusted to produce as nearly a 3-in. slump as possible. The test period for the gravel batches was from July 2, 1959, through Aug. 11, 1959; the limestone batches from Sept. 1, 1959, through Nov. 6, 1959.

Tables B and C summarize the data of Series A showing the relationship of coarse aggregate size to compressive and flexural strength, mixing water requirements, air content, fineness modulus of total aggregate, and b/b_o ratio.

As was expected, the water requirement was reduced as maximum size of coarse aggregate increased. However, in spite of reduction in the water-cement ratio due to decreased water demand, strength of the concrete did not continue to improve as maximum size aggregate increased.

Fig. B through E show graphically the effect of changing the size of coarse aggregate on compressive and flexural strengths. In nearly all cases, where coarse aggregate size was increased above 1 in., both compressive and flexural strengths of the mixes were reduced.

TEST SERIES B

In the earlier study on the effect of coarse aggregate size on strength of concrete, a wide range of mixes was covered and only a few cylinders tested for each type batch. Only single batches of a given mix proportion were prepared. This program expanded the earlier study with the number of batches and specimens increased to provide greater assurance of accuracy and to reduce the possibility of a single specimen seriously effecting the results.

For the purpose of this series, a single cement content was used (6.25 sacks per cu yd) with three gravel and three crushed limestone coarse aggregate gradings. These gradings shown in Table D correspond to ASTM C 33-57 size numbers 357, 467, and 57 and are designated by their maximum sizes as 2, 1½, and 1 in., respectively. For each of the six coarse aggregate sizes, one plain, one retarded, and three air-entrained batches were produced.

From the air-entrained mixes, six 6 x 12-in. cylinders, two 6 x 6 x 24-in. beams, and one 3 x 4 x 16-in. freeze-thaw specimen were made from each batch. For the plain and retarder mixes, six 6 x 12-in. cylinders, three 6 x 6 x 24-in. beams, and one 3 x 4 x 16-in. freeze-thaw specimen were made.

Cement was Type I. Aggregates were White Marsh concrete sand, White Marsh gravel, and Texas crushed limestone. Air content was based on ACI 613-54 for air-entrained and plain mixes. Slumps ranged from 2

to 3 in. Calcium-lignosulfonate admixture was used at the rate of 2 oz per sack of cement in the retarded concrete mixes. Water content for these mixes was based on information obtained from our previous test data which supplements the standards set up in ACI 613-54. The air-

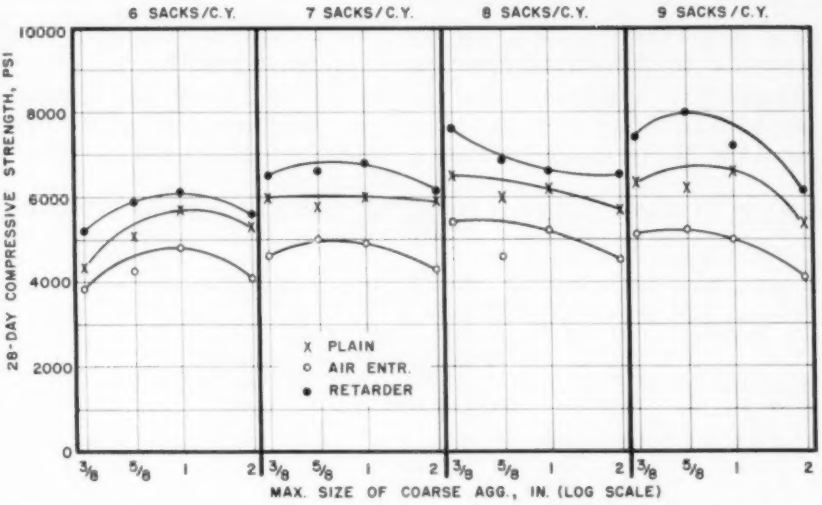

Fig. B—Effect of size of gravel coarse aggregate on compressive strength (Series A)

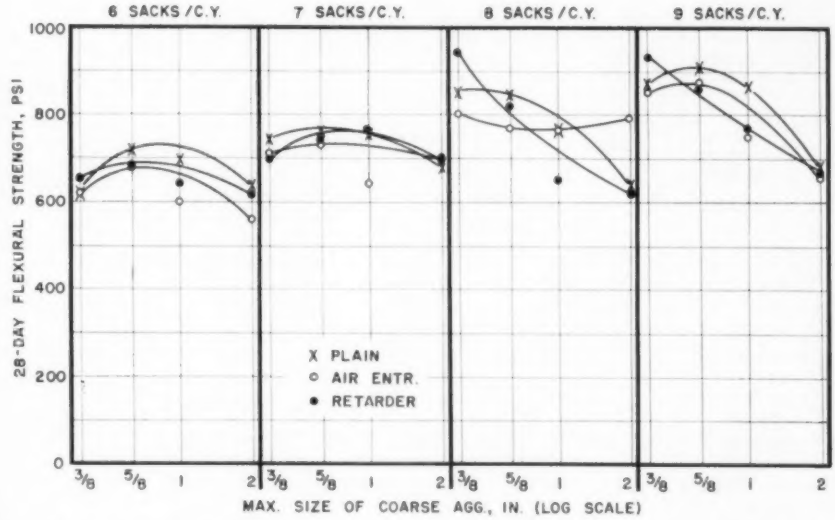

Fig. C—Effect of size of gravel coarse aggregate on flexural strength (Series A)

entraining agent was proportioned at the rate of 0.4 oz per sack of cement. The test period for the study was from Apr. 27, 1960, through June 7, 1960, except for the durability studies.

Table E summarizes the characteristics of each mix. When more than one batch of a given mix was made, as in the case of the air-entrained mixes, results were averaged.

Fig. F shows graphically the comparison between compressive and flexural strengths and maximum size of coarse aggregate.

Freeze-thaw specimens were molded in steel molds and carefully covered with saturated burlap so that no moisture was permitted to be lost. After 24 hrs, specimens were stripped from molds and placed in a

TABLE D — CHARACTERISTICS OF AGGREGATES AS USED IN CONCRETE, SERIES B

Gravel

Sieve size or item	Coarse aggregate maximum size			Fine aggregate
	1 in.	1½ in	2 in.	
	Grading, cumulative percents passing each size			
2½ in.	—	—	100.0	—
2 in.	—	100.0	95.3	—
1½ in.	100.0	92.5	71.45	—
1 in.	95.2	79.22	51.65	—
¾ in.	78.4	55.20	41.20	—
½ in.	51.5	29.82	26.90	—
⅜ in.	32.1	12.88	17.00	100.0
No. 4	5.2	1.20	2.60	96.7
No. 8	2.6	0.30	1.30	83.7
No. 16	—	—	—	71.6
No. 30	—	—	—	52.8
No. 50	—	—	—	17.3
No. 100	—	—	—	2.3
Fineness modulus	6.82	7.33	7.62	2.76
Specific gravity bulk SSD	2.64	2.63	2.63	2.63

Stone

Sieve size or item	Coarse aggregate maximum size			Fine aggregate
	1 in.	1½ in	2 in.	
	Grading, cumulative percents passing each size			
2½ in.	—	—	100.0	Same
2 in.	—	100.0	93.6	as
1½ in.	100.0	100.0	65.9	above
1 in.	99.6	89.9	51.0	
¾ in.	90.0	56.2	46.2	
½ in.	62.6	30.25	32.1	
⅜ in.	30.1	15.6	15.1	
No. 4	5.1	4.0	2.6	
No. 8	4.2	3.5	2.1	
No. 16	—	—	—	
No. 30	—	—	—	
No. 50	—	—	—	
No. 100	—	—	—	
Fineness modulus	6.71	7.21	7.68	
Specific gravity bulk SSD	2.82	2.82	2.82	

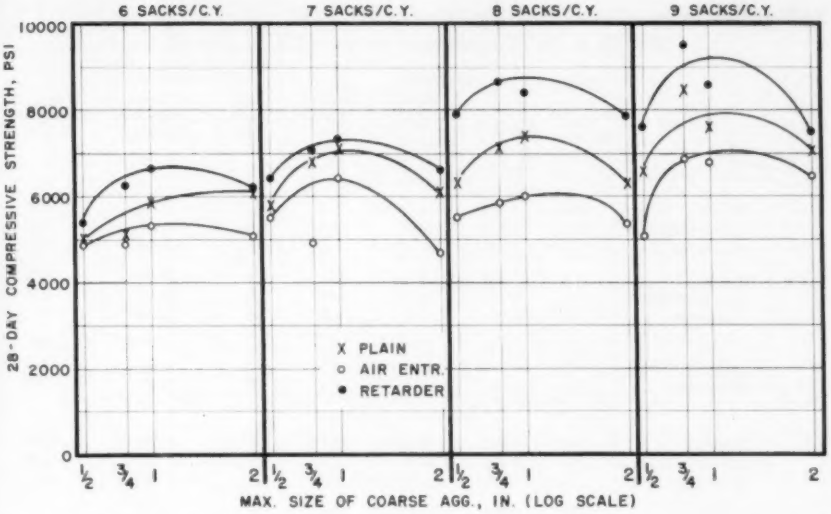

Fig. D—Effect of size of stone coarse aggregate on compressive strength (Series A)

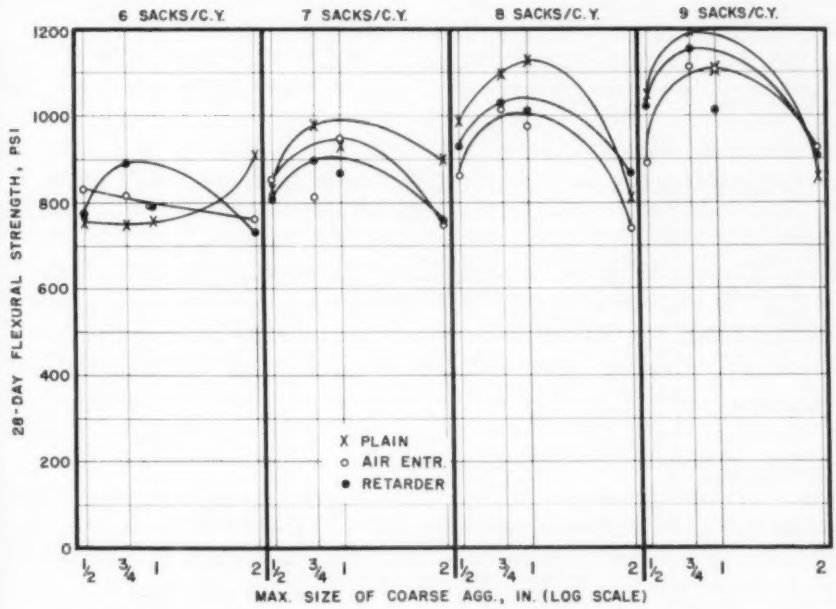

Fig. E—Effect of size of stone coarse aggregate on flexural strength (Series A)

TABLE E— CHARACTERISTICS OF FRESH AND HARDENED CONCRETE, SERIES B (6.25 sacks per cu yd—2-3-in. slump)

Maximum size aggregate, in.	Type concrete	Aggregate proportioning: F.M. total aggregate*	Aggregate proportioning: b/b_o†	Aggregate proportioning: Sand, percent total aggregate	Mixing water, gal. per sack	Percent air	Compressive strength 28 days‡	Flexural strength 28 days§
		Group 1—Gravel						
1	Plain	5.45	0.721	35.4	5.3	2.1	5577	835
	Air-entrained	5.44	0.714	34.0	5.0	4.6	4681	789
	Retarder	5.34	0.700	36.4	5.1	4.4	5864	892
1½	Plain	5.60	0.754	33.6	5.3	1.2	5309	800
	Air-entrained	5.88	0.763	31.8	4.8	3.9	5087	777
	Retarder	5.80	0.759	33.4	4.9	2.9	6045	817
2	Plain	6.17	0.790	29.9	4.9	1.6	4828	833
	Air-entrained	6.25	0.793	28.2	4.8	3.8	4337	703
	Retarder	6.15	0.787	30.2	4.6	3.0	5217	760
		Group 2—Limestone						
1	Plain	5.06	0.671	41.8	5.7	1.9	6250	1047
	Air-entrained	5.15	0.672	39.4	5.5	5.1	5482	836
	Retarder	5.15	0.669	39.4	5.3	4.2	6932	917
1½	Plain	5.46	0.718	39.4	5.9	1.5	6515	897
	Air-entrained	5.59	0.738	36.4	5.4	3.8	5767	865
	Retarder	5.52	0.733	38.0	5.3	3.4	7095	932
2	Plain	5.80	0.731	38.3	5.6	2.2	5588	901
	Air-entrained	5.95	0.754	35.1	5.1	4.1	5415	833
	Retarder	5.89	0.748	36.4	4.96	4.0	6106	939

*Fineness modulus of combined fine and coarse aggregates.

†Ratio absolute volume of coarse aggregate per unit volume of concrete to absolute volume of coarse aggregate per unit volume of dry rodded coarse aggregate (equivalent to dry rodded volume of coarse aggregate per unit volume of concrete).

‡Each value average of three cylinders for plain and retarder mixes and nine cylinders made from three separate batches for air-entrained batches.

§Average of three beams—one beam from each of three separate batches. All others average of three beams from single batch.

TABLE F— SUMMARY OF FREEZING AND THAWING TEST DATA, SERIES B

Type of concrete	Maximum size of coarse aggregate, in.	No. of beams averaged	Durability factors*: At 200 cycles: Gravel	At 200 cycles: Stone	At 500 cycles: Gravel	At 500 cycles: Stone
Air-entrained	2	3	97	99	97	104
	1½	2 or 3	99	106	99	112
	1	3	99	99	104	103
Retarded	2	1	98	100	99	108
	1½	1	94	101	99	48
	1	1	103	99	104	99
No admixture	2	1	†	11	†	4
	1½	1	6	7	3	3
	1	1	22	7	9	3

*D.F. $= PN/M$, where $N =$ number of cycles at which D.F. is being calculated or number of cycles when dynamic modulus of elasticity (E) reached 50 percent, whichever occurred first.
$P =$ dynamic modulus at N cycles as percent of original.
$M =$ number of cycles at which D.F. is being calculated.

†Specimen not furnished.

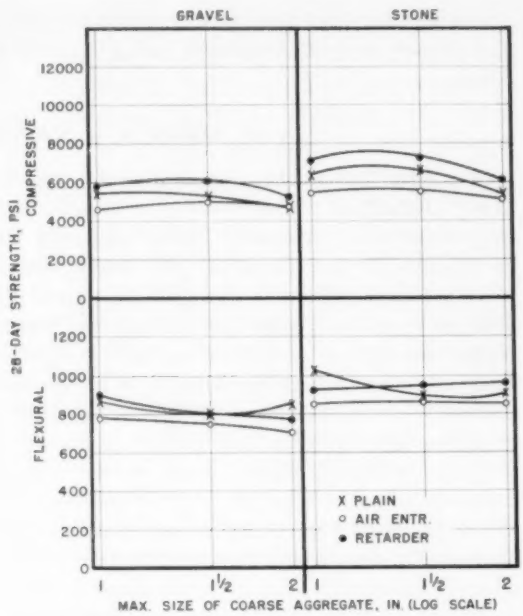

Fig. F—Effect of maximum size of coarse aggregate on compressive and flexural strength (Series B)

saturated lime-water solution. When the oldest specimen (from the first batch) was 13 days, the entire lot of specimens including those only 5 to 6 days old, was removed into cans filled with saturated lime water and carried to the freeze-thaw testing agency. At age 14 days, the beams were exposed to alternate freezing in air at 0 F and thawing in water at 40 F in automatic apparatus producing about seven cycles per day (ASTM C 291). Summary of data on durability studies is shown in Table F. From the data furnished, there does not appear to be any significant differences among the various maximum sizes or between the gravel and stone so far as freezing and thawing resistance is concerned.

In all batches, the amount of water required was greater as the coarse aggregate size was reduced. The least amount of water was in the 2 in. air-entrained coarse aggregate sizes for both gravel and stone batches. However, Table E shows that this had the lowest strength of all mixes. This again confirmed the findings of the earlier Harry T. Campbell Sons' Corp. study.

Flexural strength proved to be extremely difficult to analyze. Results were sporadic with no pattern indicated. Beams tested with the same mix, but from different batches, gave widely divergent results. The lack of uniformity of these flexural test specimens points out the undesirability of using the flexural strength as a criterion of the merit of the concrete produced.

We believe that our findings tend to confirm those of Messrs. Walker and Bloem and are indebted to the authors for opening up this field of study.

By MARVIN J. HAWKINS*

"It's hard to teach an old dog new tricks," and this may well apply to many readers of the authors' new concept of the effect of aggregate size on properties of concrete. There are many who will seriously doubt that at the same cement content and slump, concrete of small aggregate will have the same strength and quality as concrete with aggregate graded to a larger maximum size. For many years concrete engineers have used the water-cement ratio method of mix proportioning and have built up a demonstrated experience record of its general reliability for concrete of ordinary strength. The use of the largest practical size aggregate in concrete with its resultant reduction in mixing water requirements and the corresponding reduction in cement requirements has been one of the primary objectives of the concrete engineer during mix calculations. Many years of laboratory research associated with the proportioning of concrete mixes, using dfferent maximum size aggregates, has demonstrated the merits of the water-cement ratio method in the compressive strength range normally used in construction. It was only natural, therefore, for some of us who design by the water-cement ratio method to question and check with comparative mixes some of the test results reported by the authors.

When the authors' preliminary report was released [37] (*JRL Publication* No. 7, National Sand and Gravel Association and National Ready Mixed Concrete Association, Aug. 1959[5]), our project laboratory made comparative tests of three mixes reported by the author. It was found the strength results of our local tests using government tested cement and aggregates followed the water-cement ratio trend instead of the results reported by the author.

Three comparative mixes were made with saturated surface-dry uncrushed aggregates, uniformily graded sand, and pre-blended Type II, low-alkali, government tested cement. The batches were mixed without entrained air, with cement contents of 6 sacks per cu yd at 2½ to 3-in. slump. The mix characteristics and resulting compressive strengths were:

Maximum aggregate size, in.	Percent sand	Cement, per cu yd sacks	W/C		Slump, in.	Trapped air, percent	Compressive strengths, psi		
			Weight	Gal per sack			7-day	28-day	90-day
1½	35*	6.02	0.47	5.3	2¾	0.17	2775	4770	6360
¾	40*	6.01	0.51	5.8	2½	0.76	1890	3585	4885
⅜	50*	5.98	0.63	7.1	2¾	0.89	1680	3315	4810

*Same percent sand as shown by authors' Table 4.

*Member American Concrete Institute, Chief, Materials Engineering Division, USBR Trinity Project, Lewiston, Calif.

Aggregates in the above mixes were predominately 100 percent basalt with individual loss in five cycles by sodium sulfate test as follows: sand, 9.3 percent; No. 4-⅜ in, 0.8 percent; ⅜-¾ in, 0.3 percent; and ¾-1½ in., 0.5 percent. The above results using project cement and aggregates tend to follow the water-cement ratio principle of mix proportioning.

Careful review of the authors' test data discloses some interesting items. Most interesting is the possibility that mix proportioning by the water-cement ratio method may have its limitations, particularly for extra high strength concrete. Their strength curves tend to follow the water-cement ratio trend in ordinary 3000-psi concretes. In the high strength concretes their strength curves show smaller size aggregate resulting in higher strengths at same cement per cubic yard. Their findings should be helpful to many prestressing firms producing special sections where extra high strength concrete is desired. We cannot, however, overlook the test data that has demonstrated the value of proportioning concrete by the water-cement ratio method, particularly for the common concrete uses. Also we should not overlook the questionable percent of sand the authors used in most of their test mixes as reported in Table 2, p. 286.

For several years, during construction of Grand Coulee Dam, considerable concrete mix research was conducted in the project laboratory. During this period one of the authors, Stanton Walker, presented his January 1940 paper, "Design of Concrete Mixtures," at the tenth annual convention of the National Ready Mixed Concrete Association. The water-cement ratio principle of mix proportioning was so well presented by the author in this paper, that several of us in the Coulee Dam Laboratory reproduced it to note book size. The author's practical approach to mix proportioning has been referred to frequently during the years.

In his 1940 report, Table 3, the author shows decreasing percents of sand as the maximum size aggregate in concrete is increased. The percent sand shown is ten less for 2-in. maximum size gravel mixes than for ¾-in. maximum size gravel mixes. This agrees closely with present-day mix proportioning practices. It is noted, however, the authors in the ACI report under discussion show they used about the same percent sand in the ¾ in., 1½ in., and 2½ in. mixes. This is shown in Table 2, Series 173. Knowing that mixing water requirements decrease with corresponding reductions in the percent of sand in concrete mixes, this effect on water-cement ratio for the mixes reported would be expected to influence test results.

It is suggested that the authors should also have included soundness test results for each size aggregate used in their comparative mix studies. The reporting of soundness for each size gravel, by either abrasion or sodium sulfate test methods, would provide some factual information for judging their quality. Individual aggregate fractions

tested and found to have equal soundness would eliminate the possibility that some breakdown may have occurred in the larger coarse aggregate fractions during mixing and testing of concrete.

Tests show that durability of concrete can be influenced by the unit water content. By using mixes with the largest maximum size aggregate we reduce unit water content. This improves the concrete durability and also reduces the harmful effects that may occur from drying shrinkage.

The use of the small aggregate size concrete mixes definitely have their place in particular locations during construction. At times we find some who request the use of ¾-in. maximum aggregate concrete mixes for all concrete. It would be well to elaborate on this subject, particularly the job site problems. The percent crushed in aggregates produced by many producers is comparatively low because large aggregate sizes make up a significant proportion of the production. Naturally the percent crushed with its resultant chips and inferior workability increases when the maximum size aggregate is reduced. This is often overlooked by a commercial producer of concrete, but not by the concrete finisher or concrete crews placing the concrete. The advantage to be gained in workability of concrete mixes, having natural or low percents of crushed, is well known to construction men responsible for concrete placement.

By J. HOLLEB*

I was glad to read in the report by Messrs. Walker and Bloem the statement that "The earlier evidence of the grading effect has been generally overlooked because of the broad and unqualified acceptance of the water-cement law." This statement is in full accordance with my opinion.

We read further in the report: "The grading of each was selected to conform to good practice and to meet conventional specification requirements." Which is to say the conventional specifications required a continuous grading in arbitrary proportions. This is not rational and does not confirm the experience.

The problem of grading and proportioning is to establish a mix giving a homogeneous concrete, with minimum friction between the particles and requiring a minimum amount of sand and water for a maximum compaction. This can be obtained more easily with a discontinuous grading of the aggregates.

The authors have used the same grading of sand for all four different sizes of aggregates and the same proportion of sand for the aggregates of ¾, 1½, and 2½ in. size. In our opinion the grading and proportions

*Chicago, Ill.

do vary with the maximum size and because of this, the results of the study by Messrs. Walker and Bloem are discutable.

The advantage of using the maximum size of aggregates is: for lean mixes to give a concrete of higher strength, for richer mixes to give a more workable concrete. The use of maximum size of aggregate is limited by the dimensions of the concrete member and the spacing between reinforcing bars. The test of concrete with 2½ in. maximum size aggregates in 6 x 12-in. cylinders is not adequate.

The generalization of the water-cement law had frozen the scientific thinking and experimentation about the problem of proportioning. The authors' study can start a break-through and they are to be congratulated for it.

By E. L. HOWARD*

Messrs. Walker and Bloem have, by their excellent research, confirmed the experience long known in the field. Ready-mixed concrete users get better strengths with smaller aggregate (¾ in. maximum) than with 1½ in. maximum, especially in the richer mixes.

Table G is taken from data gathered in 1951 for a "Letters from

TABLE G — 28-DAY COMPRESSIVE STRENGTH DATA

Number of sacks of cement per cu yd of concrete	6	6
Maximum size aggregate, in.	¾	1½
Number of cylinders	220	140
Highest psi	4950	4630
Lowest psi	2580	2900
Average psi	3960	3780

Readers" article [LR 48-2 ACI JOURNAL, V. 23, No. 1, Sept. 1951 (*Proceedings* V. 48), p. 94] and are still typical of 1960's concrete.

The test results from 24 jobs in 1960 are plotted in Fig. G. As the strength of concrete increases the supposed advantage given by larger

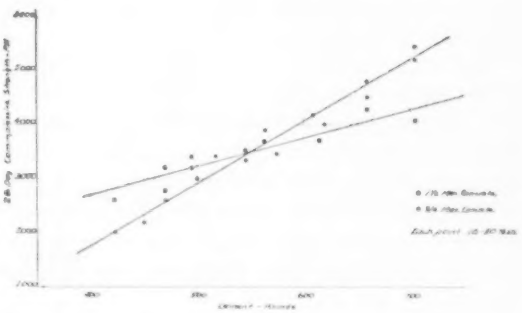

Fig. G — Comparative strengths of 1½-in. maximum and ¾-in. maximum aggregate concrete

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maximum size aggregate becomes in reality a disadvantage. Only in the very lean mixes do the 1½ in. maximum mixes yield better strengths than ¾ in. maximum mixes. Even in this range the advantage is small.

We feel that Messrs. Walker and Bloem have made a valuable contribution to the knowledge of concrete.

By M. F. MACNAUGHTON*

The information presented in this paper is extremely interesting and instructive, and the authors are to be congratulated on their presentation.

For the past 3 years or more the technical control laboratory of Mount Royal Paving & Supplies Limited, St. Michel, P.Q., Canada, has been carrying out investigations along somewhat parallel lines. Some information developed in regard to this work was presented in a paper, "The Technical Control of Ready-Mixed Concrete," before the Montreal branch of the Engineering Institute of Canada in October, 1960. Some of this information is presented here, supplemented by further data related to the problem.

The classic water-cement ratio theory offered the premise that the strength of concrete is governed by the volume relationship of water and cement in the mix. Subsequent investigations have narrowed the application of this theory very materially, and it would seem that other investigations now in progress will narrow it still more.

For example, it has generally been accepted, as a corollary to the w/c ratio theory, that with increasing maximum aggregate size, for equal cement content and equal plasticity, the water requirement would be lower, and the compressive strength correspondingly higher.

Fig. H shows compressive strength test results at 90 days on three series of trial mixes using cement and aggregates from the same sources, but with varying maximum size of aggregate particles. The curves indicate that the theory of increasing strength with decreasing w/c ratio does not always hold good when aggregates of different maximum size are under consideration.

For a more direct comparison the data from which Fig. H was derived have been plotted in Fig. I to show the w/c ratio-strength curves for the three series of trial mixes in question. These show quite clearly that, for equal w/c ratio, the mixes with the smallest size aggregate, at least in the high strength range, are the strongest.

This was puzzling, but a possible explanation was that the deleterious effects of water gain might be more pronounced with increasing size of aggregate, and the problem was dismissed without too much consideration.

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Three years ago, in carrying out strength tests at very early ages—10 to 12 hr—to determine the effects of retardation, it was found that this theory of water gain being responsible for the lower strength of the concrete with the coarse aggregate was quite untenable. Water gain, though not necessarily water loss, is, of course, complete as soon as the concrete begins to stiffen, and long before any measurable strength is developed. Yet the strengths at early ages, 24 hr or less, were quite normal, and did not exhibit any of the aberrations which develop at later ages, and which become more and more pronounced as the concrete ages and increases in strength.

To illustrate this the strengths of the mixes with the 1½ in. maximum sized aggregate, shown in Fig. H, are shown in Fig. J for ages commencing at 18 hr after mixing.

Obviously, the cause of these effects is something which develops in the concrete after it passes from the plastic to the solid state, and it is equally obvious that the most logical explanation is internal shrinkage. All of the shrinkage which occurs in concrete develops in the cement-water phase, and the aggregate is unaffected. While the concrete is still plastic, shrinkage of the cement-water phase will result in some movement of the aggregate particles, with subsequent shrinkage of the concrete as a whole. For the purpose of illustration, if we visualize a concrete consisting of only one rather large size of coarse aggregate embedded in a matrix of cement-water paste, plastic shrinkage of the mixture as a whole, accompanied by some change in the positioning

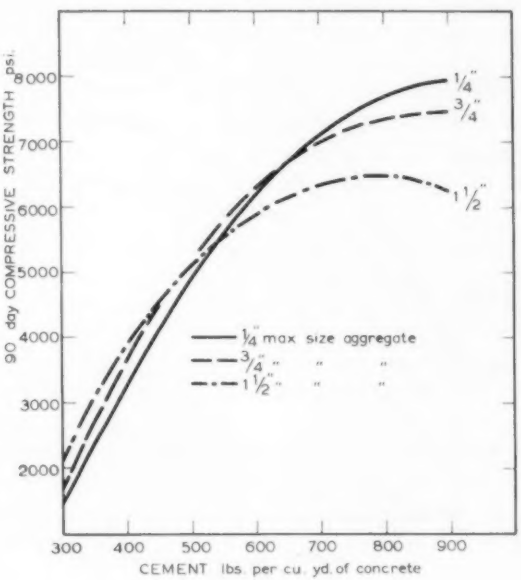

Fig. H — Compressive strengths at 90 days from three series of trial mixtures with varying maximum size of aggregate

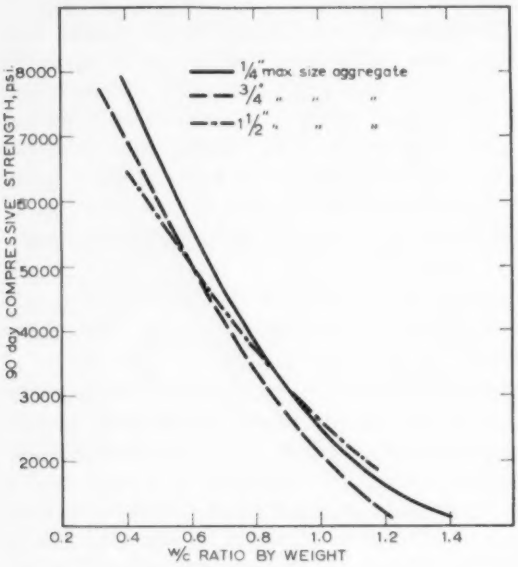

Fig. I — *W/C* and 90-day strength relationships for mixtures shown in Fig. H

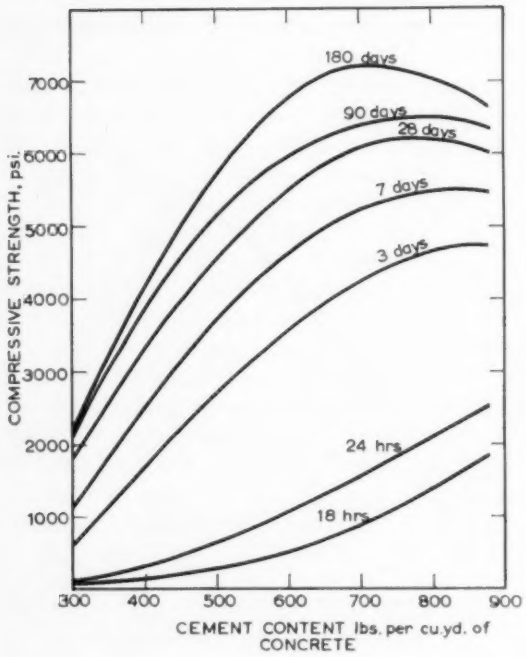

Fig. J — Compressive strengths at various ages for 1 1/2-in. maximum size aggregate mixtures shown in Fig. H

of the coarse aggregate particles while shrinkage effects develop, may take place while the concrete is still plastic, as a result of extrusion of some of the water by gravity and evaporation effects. Once the matrix has set, over-all shrinkage of the concrete as a whole is no longer possible, but shrinkage of the cement-water sandwiches between the aggregate particles still continues, and induces internal stresses which, in magnitude, must be a function of the dimension of the aggregate particles.

The greater the proportion of cement and the greater the amount of water in the mix, the greater will be the ultimate shrinkage, and the greater will be the magnitude of the locked up stresses developed in the cement paste sandwiches between the aggregate particles. Eventually, with very high cement content, and very large aggregate, these locked up stresses may become so great that the concrete may fail under the application of an external force which is actually less than that required to produce failure of a similar concrete made with: (a) less cement per unit of volume, or, (b) smaller sized aggregate.

There is a considerable volume of evidence being gathered to indicate that the above is probably the true explanation for some of the abnormalities observed in the behavior of high strength concrete with varying sizes of aggregates.

MODULUS OF RUPTURE COMPRESSIVE STRENGTH

If there is any reason at all in the above argument we should expect that, since shrinkage of the concrete after setting, results in contraction of the cement-water phase, with development, generally, of internal tensile stresses, this tendency might be more apparent in modulus of rupture tests of the concrete, as compared to compressive strength tests.

The modulus of rupture test for concrete determines the strength of the concrete in flexure, and, to some degree, is a measure of the tensile strength, since failure is induced by tension in the outermost fibers of the matrix at the lower face of the specimen.

In the compressive strength test, with shrinkage effects and internal stress development in all directions, some of the internal stresses, when the sample is loaded in compression, may be reduced. With the modulus of rupture test there is no possibility of relief of any of the internal stresses in that part of the beam which is being subjected to additional tensile stress, since all of the internal stresses, according to this theory, are originally in tension.

This is confirmed by a rather extensive investigation which was carried out in 1959 on samples from 135,000 cu yd of concrete which was used for the construction of a jet runway and accompanying taxi strips, for the Canadian Department of Transport at Dorval Airport, adjacent to Montreal.

In this investigation samples were tested for compressive strengths and comparable modulus strengths, the two sets of samples being taken from the same batch, with test results determined at 1, 3, 7, 28, 90, and 180 days.

A summary of the test results, covering 99 samples of three cylinders and three beams each, is shown in Fig. K. This shows quite clearly that age in time has nothing to do with the relationship R/f_c'. The controlling factor in this relationship is probably the magnitude of the internal shrinkage stresses which govern the strength of the concrete when tested, whether in compression or in flexure.

As can be seen, test results at all ages fall into a general pattern in which the value of R/f_c' decreases progressively as the compressive strength increases, regardless of the age of the sample. This would seem to support the premise that, as a result of internal shrinkage effects developed in the cement-water paste after the concrete hardens, internal tensile stresses are developed, and that these are reflected more vividly in modulus of rupture tests than in standard compression tests.

In Fig. K the results of all tests are plotted, first, as an average of results obtained from groups of three or more samples within a limited strength range for each age. These average results are identified by the legend. Secondly, all results, regardless of age of specimen, are segregated into groups with compressive strength falling within a range of $\pm$ 50 psi. These are also plotted as single points on the graph. It is illuminating to note that the divergence from the mean, for the average result, regardless of age, is perhaps less than the divergence of the individual groups of test results at stated ages.

Special mention should be made of the results of the tests at 24 hr. These are all below 2000 psi in compressive strength, and, because of the difficulty of maintaining these samples under standard curing conditions in the field, little attention should be given to the comparatively wide scatter of results, as compared to the results at later ages.

If we examine Fig. K closely it is apparent that, in the range of the lower compressive strengths below 1000 psi, which are of slight importance in any event, the factor $R/f_c' \times 100$ may reach a value of 19.0 to 20.0. In the higher strength range the curve seems to be tending toward progressively lower values of the above relationship, and probably, with increase of cement content and compressive strength, may show even sharper reduction of values than indicated in this series of tests, which reached a maximum compressive strength of only about 5600 psi at 180 days, with a comparatively low cement factor of 475 lb per cu yd.

At the maximum strength of 5600 psi obtained from this series of tests at 180 days the curve representing the relationship between the compressive strength and the modulus of rupture strength is veering

sharply toward increasingly lower values of this relationship with increased compressive strength of the concrete. From an analysis of the results obtained it would seem that the modulus strength, even for this mix with comparatively low cement factor, and a maximum nominal aggregate size of only 1 in., reached a ceiling at a compressive strength of about 5000 psi. Attainment of compressive strength greater than this, whether from the use of more cement in the mix, or increased aging of the concrete, probably will result in an actual decrease in the modulus strength.

ANALYSIS OF RESULTS BY WALKER AND BLOEM

An attempt has been made to analyze the results shown by Messrs. Walker and Bloem on the basis of the method shown in Fig. K. This was only partially successful, largely because the test results reported for any one class of concrete were comparatively limited in scope and in number. Fig. L shows the relationship of R/f_c' for their tests on plain mixtures with 2½-in. aggregate ranging in age from 7 to 91 days. It seems likely from Fig. L that the cement content, and, presumably, the water content, in the three mixes with different cement factors,

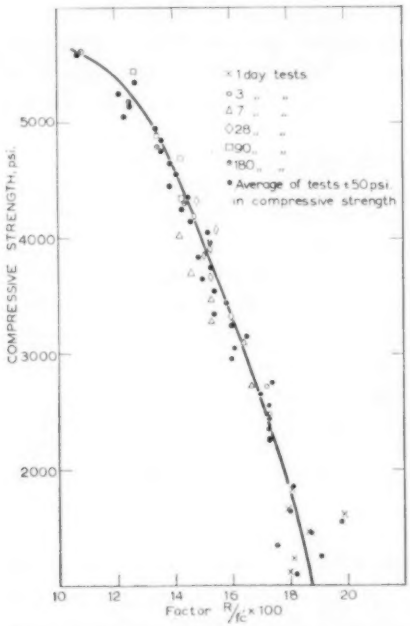

Fig. K — Modulus of rupture-compressive strength relationship derived from samples of concrete from Dorval Airport

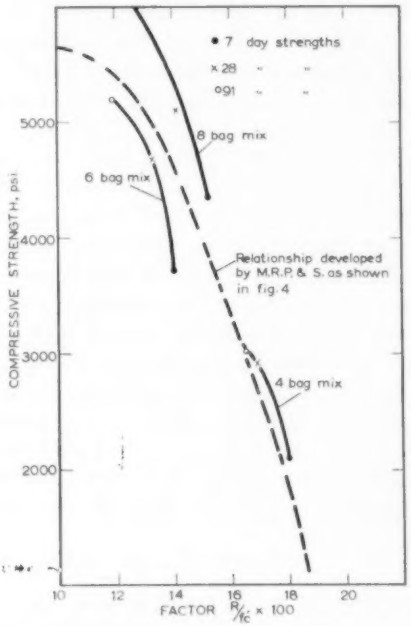

Fig. L—Walker and Bloem results from various mixtures at varying ages, with 2½-in. maximum size aggregate

materially affects the relationship. It must be remembered that in the investigations carried out by the NRMCA, both cement and aggregates were from sources different from those from which the results of the tests shown in Fig. H-K were obtained, and it is most unlikely that identical results would be obtained.

Nevertheless, in place of attempting to develop a median curve of the value of R/f_c' from the rather limited and scattered results derived from the Walker and Bloem paper, the median curve shown in Fig. K was superimposed on Fig. L. The result is surprising: this median curve, derived from test results on an entirely different series of tests on samples made with a different cement and different aggregates, and different aggregate size, fits as snugly into the over-all picture of the results reported by NRMCA as any analysis of the latter results alone might indicate. Attempts to plot, in the same manner, the results from the three other series with smaller sized aggregate reported in the paper resulted only in a wide scatter of points with many apparent inconsistencies.

SUGGESTED FURTHER INVESTIGATION

It is interesting to note that, in the Walker-Bloem paper, mention is made of the fact that analyses of the data furnished in papers dating back to 1927 showed that test results obtained indicated the occurrence of similar aberrations in the behavior of concrete with aggregates of varying maximum size, even though the significance of such results was not appreciated at the time.

Now that the problem has been brought out into the open as a result of the publication of the Walker and Bloem papers it would be desirable to carry out additional tests, on a much broader scale, to inquire further into the basic principles underlying the performance of mixtures with varying sizes of aggregate. Such an extensive investigation is probably beyond the capacity of NRMCA, and, most certainly, is beyond that of Mount Royal Paving & Supplies Limited, to undertake. It might be fathered by one of the major organizations interested in the development of concrete technology on this continent. Certainly, if one-quarter as much effort and expenditure had been made in developing concrete technology in the past decade as has been devoted to putting dogs and monkeys into orbit we would know far more about the behavior of concrete than we do today.

By A. R. MEAD*

Messrs. Walker and Bloem have made an important contribution to the published information on the effects of the maximum aggregate size on the strength and shrinkage properties of concrete. The U. S. Army

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Engineer District, Sacramento, Calif., has permitted contractors on some projects an option of using certain maximum aggregate sizes; on other projects the maximum size of the aggregate has been specified. The cost of obtaining a suitable strength, durability, or other property of the concrete has usually been the controlling criterion. Our strength data corroborates Messrs. Walker and Bloem's data; however, the results are interpreted differently.

There is an optimum cement factor for each series of mixes of equal workability and the same consistency, which uses a specific aggregate of a certain maximum size, at which the unit amount of mixing water required will be a minimum (Fig. M). The slumps of the laboratory trial mixes from which these data were obtained varied from 1¼ to 3¾ in. and were adjusted to a 3-in. slump by the highly questionable assumption that adding a gallon of water to the cubic yard batch would increase the slump 1 in. The erratic data may be due to this adjustment.

There is another optimum cement factor, for each series of mixes using a specific aggregate of a certain maximum size, at which the unit strength of the concrete divided by the pounds of cement in a cubic yard batch will be a maximum. Typical graphs of the 28-day compressive strengths divided by the pounds of cement in a cubic yard batch are also shown on Fig. M. The data are from the laboratory trial mixes previously referred to and were made for the purpose of selecting job mixes to fit different requirements. The different series of trial mixes and the different types of molds used to cast the

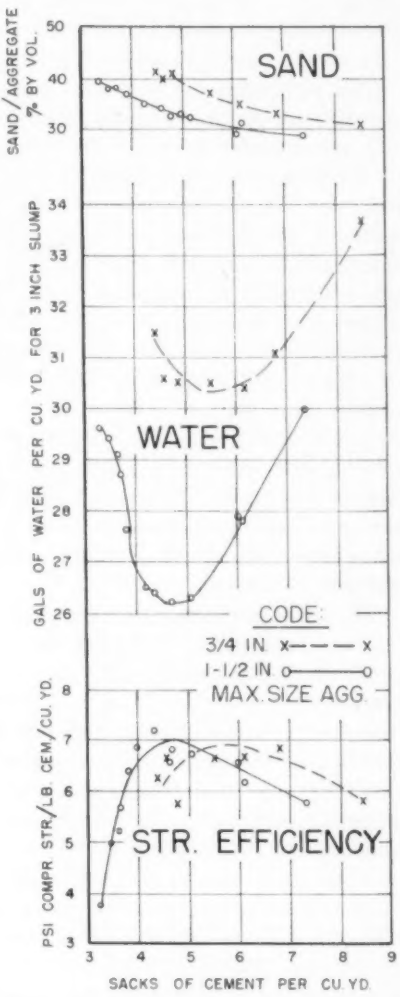

Fig. M—Effects of ¾-in. and 1½-in. maximum aggregate size on sand-aggregate ratios, amounts of mixing water, and strength efficiencies for air-entrained mixes of equal workability and consistency

test cylinders introduced variables which caused the erratic data. The aggregate was a river sand and gravel which consisted of rounded to crusher processed angular particles of hard, durable meta-volcanics, meta-sediments, amphibolite, and meta-basic-igneous rock types with minor amounts of basic igneous, granitics, vein quartz, and volcanic types. A Type II portland cement from one of the mills in northern California was used in all mixes. A neutralized vinsol resin admixture was used to entrain about 4 percent air by volume of the entire mix.

The optimum cement factor for the best strength efficiency of a specific series of concrete mixes is a variable affected principally by (1) the amount of water in the mix, (2) the amount of entrained air in the mix, and (3) the age at which the specimens are tested. The data indicate that the optimum cement factor for the least amount of mixing water is about equal to the optimum cement factor for the best 28-day compressive strength efficiency; however, our experiences indicate that the effects of entrained air and the age at which the specimens are tested may cause considerable differences between the optimum cement factors for the best strength efficiency and for the minimum amount of mixing water. The best 7-day strength efficiency is developed at a higher unit cement content than the optimum cement content which develops the best 28-day strength efficiency. The effect of testing at later ages is to decrease this optimum cement content. In theory, there will be an optimum cement factor for the best strength efficiency in any series of mixes conforming with Abrams' water-cement ratio law, even when the amount of mixing water per unit volume of concrete does not vary.

The strength efficiency per pound of cement is influenced by all of the variables which affect the strength of the concrete, but for the purposes of this discussion they may be reduced to (1) the unit amount of cement in the mix, (2) the unit amount of water in the mix, (3) the percentage of air in the cement-air-water matrix, and (4) a variable relating to the physical and chemical properties of progressively larger size fractions of coarse aggregate on the strength of the concrete. The effect of increasing the maximum size of the aggregate usually results in better strength efficiencies for optimum cement content mixes. This increase in strength efficiency for the larger aggregate sizes has been most noticeable in the 6-in. maximum size aggregate mixes. The high strength efficiencies are attributed to the smaller quantities of mixing water; e.g., the 6-in. maximum size aggregate mix used for the lean, air-entrained concrete in the interior of Folsom Dam required 141 lb, or 16.9 gal., of water per cu yd at the optimum cement content of 2.5 sacks per cu yd. If a 1½-in. maximum size aggregate mix of equivalent strength had been used in Folsom Dam, the additional cement which would have been required in such an under-optimum mix would have increased the cost about $1½ million. Conversely, costs have been

TABLE H — CHARACTERISTICS OF AGGREGATES USED IN CONCRETE

Sieve size	Coarse aggregate maximum size, in.*			Group I fine aggregate† used / crushed granite	Group II fine aggregate used / gravel
	3/8	3/4	1 1/2		
Grading, cumulative percents passing each sieve					
1½ in.			100		
1 in.			72		
¾ in.		100	55		
½ in.		63	35		
⅜ in.	100	39	22	100	
No. 4	0	0	0	99.0	98.4
No. 8	0			85.8	83.8
No. 16				74.8	70.7
No. 30				47.3	44.3
No. 50				14.2	13.1
No. 100				3.0	2.7
F. M.	6.00	6.61	7.23	2.76	2.87
Miscellaneous tests					
Unit weight, lb per cu ft					
Granite	93.1	98.9	102.3		
Gravel	98.4	104.2	109.8		
Voids, percent	43.5	40.0	36.5		
	39.6	36.3	32.8		
Specific gravity bulk, SSD					
Granite	2.65	2.65	2.65	2.62	2.62
Gravel	2.61	2.62	2.62		

*Coarse aggregate separated and recombined as shown.
†Fine aggregate a combination of three sands. Grading average of two tests.

reduced by using a near-optimum 1½-in. in lieu of an above-optimum 2½-in. maximum size aggregate mix for concrete placed in certain airfield pavements.

The data are interpreted as indicating optimum maximum sizes of aggregate in concrete mixes. If the cement content of a mix is below the optimum, better strengths and cement economies may be obtained by using a larger size aggregate. If the cement content is above the optimum, better strengths and cement economies may be obtained by using a smaller size aggregate. The determinations may be made by noting the trends of the unit strengths per unit amount of cement. Variations in the amounts of mixing water to provide the desired mix consistencies may also be a consideration.

By HARRY H. MITCHELL*

The discussions of the effects of aggregate size on the properties of concrete by Messrs. Walker and Bloem have been of great interest to us because if their results applied to our aggregates, a savings of approximately $0.50 per cu yd could be passed on to the construction industry when the small size aggregates were used.

*Member American Concrete Institute, Vice-President of Research and Development, Spartanburg Concrete Co., Inc., Spartanburg, S.C.

TABLE I—CHARACTERISTICS OF FRESH CONCRETE

Batch No.	Maximum size aggregate, in.	Calculated cement, sacks per cu yd	Mixing water, gal.		Aggregate proportioning			Slump, in.	Weight, lb per cu ft	7-day avg of 2 cylinders, psi	28-day avg of 2 cylinders, psi	7-day three batch avg, psi	28-day three batch avg, psi
			Per cu yd.	Per sack	F.M. of total aggregate	b/b_o	Sand percent total aggregate						
Group I—Crushed granite													
1	3/8	5.80	42.3	7.3	4.56	0.655	44.5	2	143.4	2960	4430		
2	3/8	5.80	42.2	7.3	4.56	0.655	44.5	1	143.6	3065	4650	2995	4545
3	3/8	5.95	43.5	7.3	4.55	0.648	44.7	2	143.5	2960	4560		
4	3/4	5.85	39.4	6.7	5.04	0.718	40.9	2½	146.5	3635	5145		
5	3/4	5.87	38.8	6.6	5.04	0.721	41.0	2	146.8	3670	5165	3660	5165
6	3/4	5.85	38.6	6.6	5.04	0.720	40.7	2	146.1	3675	5180		
7	1½	5.88	38.7	6.6	5.56	0.705	37.1	4	147.6	3780	5305		
8	1½	5.96	35.6	6.0	5.61	0.715	38.4	1½	148.2	4135	5755	3985	5560*
9	1½	5.90	36.6	6.3	5.56	0.708	37.2	2¼	147.5	4045	5685		
Group II—Gravel													
10	3/8	5.97	40.7	6.8	4.68	0.666	42.0	2	145.1	3745	5235		
11	3/8	5.98	39.9	6.7	4.68	0.666	42.0	1½	145.0	3845	4960	3780	5145
12	3/8	5.95	39.6	6.7	4.68	0.662	42.0	3	144.3	3745	5240		
13	3/4	5.96	35.6	6.0	5.19	0.687	38.0	1¼	147.8	4485	5400		
14	3/4	5.96	35.6	6.0	5.19	0.687	38.0	1¼	147.7	4320	5285	4310	5365
15	3/4	5.95	35.5	6.0	5.20	0.686	37.9	3	147.2	4130	5410		
16	1½	5.98	33.8	5.7	5.66	0.685	36.0	1½	149.0	4375	5225		
17	1½	5.97	33.7	5.6	5.66	0.685	36.0	1½	148.9	4260	5455	4255	5320
18	1½	5.96	33.7	5.7	5.66	0.685	36.0	3½	148.6	4125	5285		

*5720 if Mix No. 7 is disregarded.

Laboratory mixes were made, duplicating Messrs. Walker and Bloem's tests as closely as possible. The first group used our regular coarse aggregate, a crushed granite (Table H). The results (Table I) are typical of a w/c ratio curve.

Group II tests were made using Lilesville, N. C., gravel (Table H). The results (Table I) show somewhat the same results as Messrs. Walker and Bloem's tests in that the maximum strength occurred with the ¾-in. maximum size.

Possibly the reasons may be related to the bond of the mortar and the rock and the total surface area involved. However, it is apparent from these tests and others that each type of aggregate has its own characteristics and that for an intelligent use of any given aggregate, more should be known than just its gradation and specific gravity.

By M. SPINDEL*

The research described in this paper, supplementing earlier work, gives results which apparently are in contradiction with the views obtained in theory and practice by many leading concrete experts. It has to be admitted that the authors were aware of this fact and that their investigations and tests were made with carefulness according to standards, under the guidance of experts with great experience on aggregates and concrete.

Nevertheless, the statement in the summary and conclusion of the paper that "the strength of concrete was progressively reduced as maximum size was increased above ¾ in., or in most cases above ⅜ in.", cannot be accepted generally and it is to be hoped that the authors, in further research, will find out as to what special circumstances these results were due.

There are many differences between various aggregates depending on their origin, chemical, petrographical and physical properties, strengths, resistance to weather and agressives, whether they come from deposits in dry land or in water, whether already found in the necessary sizes or crushed, cleaned and graded, etc. These differences are, of course, much greater and more complicated than the differences in modern portland cements from various plants.

In whatever circumstances the aggregates are obtained the maximum size, shape, texture, and grading are the most important characteristics, and the grading is usually considered the most essential. As the writer pointed out in a previous discussion† there were already in 1904 in Europe, government standards for concrete demanding that as much cement had to be added as was necessary to fill completely the measured

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†Spindel, M., discussion of "Effects of Incomplete Consolidation on Compressive and Flexural Strength, Ultrasonic Pulse Velocity, and Dynamic Modulus of Concrete," ACI JOURNAL, V. 32, No. 3, Part 2, Sept. 1960 (*Proceedings* V. 56), pp. 1463-1465.

voids in sand and as much cement and sand had to be added to fill completely the voids measured in coarse aggregate.

The writer, when responsible for the concrete in the Bridge Office of State Railways, was sorry to find that these standards did not and could not work satisfactorily, and that some more reliable methods had to be used to get the results desired. As mentioned in his discussion referred to before, the writer's method published in 1913 was based on mathematical calculation of the diameter of the particles (a) which could lay in the voids of compacted particles of larger size and the diameter of particles and (b) which could get through the voids of compacted particles of larger size. The result was for the smaller size to be for (a) 1/5 and for (b) 1/7 of the larger size, both calculated as balls. This calculation is, in principle, still valid both for gap grading and continuous grading and is still used by some concrete makers.

Again, from the very beginning various grading curves were developed for concrete aggregates. One of the first grading curves was published by Will B. Fuller and Sandford E. Thompson* and this Fuller curve was frequently used in many countries. Later some more grading curves were published until the latest published in 1959, which was based on a lot of mathematics and was not accepted.

The most successful grading for aggregates was introduced by the fineness modulus of Abrams,[4] which formula for well graded aggregates gave a fineness modul usually in good relation to the water-cement ratio needed for concrete of sufficient workability as tested by the slump test. This was supplemented by a formula which gave the compressive strength of concrete depending on the water-cement ratio only.

Both the fineness modulus by Abrams and the slump tests, although of advantage and used all over the world and also in the tests made by the authors, had some imperfections:

1. The slump test, although the simplest way of getting workability on the job, was found to be often an unreliable test even for wet concrete and quite impossible for very dry mixes of low workability. Therefore, it had to be, and was often, replaced by different types of tests, especially vibrating tests such as used by Powers in the United States and the latest of which is the Swedish VEVB "consistometer" used for research and in practice.

2. The fineness modulus by Abrams is obtained by sieving the aggregate on ten sieves of different sizes of mesh and adding the weights of the residues on these sieves divided by 100. Apart from the fact that nobody knew the meaning of this modulus, everybody was dependent on the special meshes of the ten sieves used in the United States by Taylor to get the correct results.

3. The Abrams' formula that the strength of concrete was dependent on the water-cement ratio only was often the reason that insufficient water was used for concrete, giving much reduced strength owing partly to the many air voids resulting from insufficient water.

*Engineering News-Record, May, 1907.

The writer, as a member of the engineering committees dealing with these questions, succeeded in explaining the meaning of the fineness modulus by Abrams in finding that it was an average of the quantities of the different sizes between the ten sieves and thus it could easily be dealt with in one part of his "fourmatter parallelogram," in a way that the same result could be obtained with sieves of different mesh sizes. In his paper on "Graphic Representation of the Composition of Concrete and Aggregates in the Fourmatter Parallelogram"* the writer showed the meaning of the Abrams' fineness moduls and how to handle it easily, even to combine mixes of different fineness modulus, to get the aggregate suitable for the purpose in question.

As mentioned before, the slump test has often been replaced by vibrating tests, especially for research work and important jobs, but there is still one matter of testing aggregates often much neglected and that is to realize that sieving on sieves alone, as a rule, does not give much information on the shape and texture of the particles of aggregate, whether very small or large. For instance, the flat shape of mica and its texture is the cause of highly reduced strength and the writer always advised to have the mica particles blown away if appearing in certain cracked stone, or to be washed away by flotation if found in rivers or in wet condition. This has also been done at the largest dams built in Europe.

It is well known how harmful very fine sand in concrete can be but there is still left open the question about the results showing the disadvantages of large size aggregates. Perhaps one reason might be that the shape and grading of the aggregate used by the authors was not favorable to sufficiently compacted concrete in molds of 6-in. diameter or even of 8-in. diameter as were used before by one of the authors. In Europe at the very beginning there were standardized test cubes of 12 in., so that larger pieces of aggregate were not prevented from settling in the best way possible. But this could perhaps explain the reduced strength of the aggregate of 2½ in., but not of 1½ in. or even ¾ in. according to the paper.

Much more probable for getting this results is that 8 bags of cement per cu yd are far too much for concrete with aggregate up to 2½ in. and to the writer's knowledge these rich mixes are seldom used for such large aggregate size concrete. It appears to the writer that the very rich mortar, perhaps in larger dimensions, between the large pieces of aggregate, and rather wet, had to shrink both during setting and hardening thus causing internal stresses or even some very fine cracks between mortar and the large pieces of aggregate which reduced the strength, this even more so after 90 days of hardening because of further shrinkage.

*Beton und Eisen (Berlin), No. 1, Jan. 5, 1931; No. 2, Jan. 20, 1931; supplemented in *Beton und Eisen*, June 20, 1931.

This should show again the importance of microscopical examination of concrete in laboratories often suggested by the writer in the ACI JOURNAL.* The microscopical examination of concrete in research laboratories will give, much more than the usual tests, information about the advantages of concrete for structures of every type and especially about the advantages to be obtained with suitable admixtures, to which, of course, air entrainment also belongs, as tested by the authors.

As to the largest size of aggregate used in practice, it may be mentioned that it depends mainly on the size of the structure in question and especially on the size available between forms or between the steel in reinforced concrete, etc. In Great Britain the usual maximum size of aggregate for concrete is still ¾ in., but at the large dams built on the continent of Europe aggregate sizes up to 5-6 in. were used successfully with the most suitable cement factors, grading curves, and admixtures for the purpose in question obtained by the latest methods of testing.

By SVEN THAULOW†

The tests described have shown that little or nothing is gained by using large sized aggregate in ordinary concrete. This conclusion has perhaps been met with more amazement in Europe than in the United States. But there is a paradox. If the tests by Messrs. Walker and Bloem had been made in Europe with cube shaped test specimens, they would probably have confirmed the axiom that greater aggregate size does yield higher apparent strength if the water-cement ratio is constant. The reason for this is that the cube-cylinder ratio of apparent strength increases with the fineness modulus of the aggregate (i.e., greater aggregate size). This has been shown, for example, by Gyengo.[30] His test results with a constant water-cement ratio are shown in Fig. N. The writer has also run tests which have shown that the apparent strength of a cube is relatively higher than that of a cylinder when the aggregate is coarser meshed.

It is generally known also that the cube-cylinder strength ratio is manifestly effected by the relative strength of the concrete. This ratio increases inversely with the strength of the concrete.[31]

The reason that the cube strength is greater than the cylinder strength is that the friction on the loading surfaces of the test specimen acts to constrain the lateral deformation of the specimen. The apparent strength of the specimen is thus increased. This writer believes that the tendency of the frictional forces to increase the strength must primarily be attributed to the grading of the aggregate. In a concrete

*Spindel, M., *op. cit.*

†Member American Concrete Institute, Norsk Cementforening, Oslo-Blindern, Norway.

with coarse fragments of aggregate and in a lean concrete with little binder, where the aggregate particles lie close to each other, the resulting transference of forces between the aggregate particles will increase the effect of the frictional forces.

The higher apparent strength of low test specimens because of the increased friction on the loading surfaces must to a great extent be ascribed the shape of the test specimen and not the properties of the concrete. Tests[32,33] in which the friction on the loading surfaces were varied and partly eliminated, have shown that the effect of friction is virtually eliminated when the test specimen is sufficiently tall, to wit, twice the diameter.

Actually, two different testing methods are involved when concrete strength is determined cylindrical and by cubic specimens. The cube-cylinder ratio is so variable that there is a danger of drawing different technological conclusions according to the shape of the test specimen which has been used. The experiments by Messrs. Walker and Bloem are perhaps a striking proof of this. It is therefore gratifying to know that two European organizations, RILEM (International Union of Testing and Research Laboratories for Materials and Structures) and CEB (Comité Européen du Béton), have moved to give the cylinder preference as a test specimen in fundamental concrete research. These organizations cooperate with the American Concrete Institute. It is therefore to be hoped that the significance of this question will be recognized in all countries, and that the use of the 6 x 12-in. cylinder will become universal.

The tests conducted at the University of Maryland will have future significance for concrete technology. If the maximum aggregate size can be reduced, the following advantages will accrue:

(1) Workability of the concrete will be improved and there will be less likelihood of defects and pockets in the concrete.

(2) There will be less wear on concrete mixers and equipment.

(3) Watertightness of the concrete will be improved, the danger of bleeding and separation under larger fragments of aggregate and reinforcing having been reduced.

(4) It is known from experience that when the maximum aggregate size is reduced, the concrete is better able to resist temperature variations, e.g., freezing and thawing. This is probably due to a decrease in the internal stresses which are caused by the different thermal properties of the binder and aggregate.

Messrs. Walker and Bloem mention also some tests in which the tensile strength of the concrete was determined by the so-called split tensile strength test. On p. 292 they state: "A further item of interest is that the range in strength displayed by the splitting test was much lower than for the flexure test, thus affording a less sensitive measure of concrete quality." It is true that the curve for the split tensile

strength flattens out considerably more than that for the flexure test, and the latter is apparently less sensitive. But this writer cannot accept this as an argument for the tensile flexure test. Many experiments have shown that this test yields considerably higher strengths than the actual tensile strength of the concrete, which can be equated with the split tensile strength with reasonable approximation.

An ideal elastic material is assumed in computing both the flexure tensile and the split tensile strength. If concrete were such an ideal material, both tests should give approximately the same result. The fictional difference is due solely to the plasticity of the concrete, this being a much greater source of error in computing the flexure tensile strength than in computing the split tensile strength.

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32. Hansen, H.; Kielland, A.; Nielsen, K. E. C.; and Thaulow, S., "Compressive Strength of Concrete—Cube or Cylinder," *Nordisk Betong* (Stockholm), No. 4, 1960, pp. 305-320.

33. Thaulow, Sven, "Apparent Compressive Strength of Concrete as Affected by Height of Test Specimen and Friction between the Loading Surfaces," *Nordisk Betong* (Stockholm), No. 4, 1960, pp. 321-324.

By BAILEY TREMPER*

The authors have greatly clarified the implications of previous reports on this subject by including data for cement contents of 4, 6, and 8 sacks per cu yd. The inclusion of results of drying shrinkage tests is also valuable.

The data now show that the effect of aggregate size on strength in 6- and 8-sack concrete is reversed in 4-sack concrete. The curves of Fig. 7 indicate that in 5-sack concrete the maximum size of aggregate produces little or no change in compressive or flexural strength. It is also shown that in 6-sack concrete, ¾-in. aggregate nearly always produced higher strengths than did 1½-in. aggregate.

Tests performed by the California Division of Highways using two sand and gravel aggregates produced commercially in the state do not confirm the authors' findings. The test results are summarized in Tables J and K. For convenience, the strengths are reported in percentages relative to those of concrete containing 1½-in. aggregate. It will be noted that in no case did the strength produced with ¾-in. aggregate

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equal that of the 1½-in. aggregate. It will also be noted that the relative differential in strength was less in the 6-sack concrete, a trend that is in line with the authors' findings.

The reason why the California results differ from those reported by the authors may lie in the character of the aggregates being tested. It is noted that the coarse aggregate used by the authors was composed of uniform quartz and quartzite gravel. If this implies that the larger and smaller particles had equal intrinsic concrete making properties, such a condition may not always obtain with aggregates composed of a greater variety of rock types and which may be affected by natural processes of weathering to different degrees. The aggregates used in the California tests were natural sands and gravels composed of a variety of rock types. It is reasonable to assume that the weaker particles were present in greater proportion in the smaller sizes. It would seem therefore, that the authors' conclusions should not be considered to have universal application.

The relationships established by testing laboratory specimens which have been consolidated by a standard procedure which may or may not be more effective than the various methods used by the construction industry, warrants investigation with respect to performance in the latter case. The California Division of Highways has obtained some information on this phase.

On a recent paving project using 5-sack concrete which was constructed by a slip-form method which depended on a vibrating tube for consolidation, the contractor asked for, and was granted permission, for the experimental use in a portion of the work of ¾-in. aggregate in place of 1½-in. size which was specified. Some 50 cores were cut from each of the sections containing the different aggregate sizes. The cores were tested for compressive strength at ages varying from 7 to 49 days. Equivalent strengths at 28 days have been estimated to be as follows:

Maximum size of aggregate, in.	Compressive strength of 5-in. diameter cores, psi	Relative
1½	3020	100
¾	2330	78

It will be noted that the relative reduction in strength resulting from the use of ¾-in. aggregate was greater than the laboratory tests reported in Table J indicate.

A portion of the pavement containing ¾-in. aggregate was constructed with 6-sack concrete in lieu of 5-sack concrete referred to above. Core strength calculated to the age of 28 days averaged 3240 psi. These results indicate that about 5¾ sacks of cement was required to equal the strength of 5-sack concrete containing 1½-in. aggregate.

TABLE J

5-sack concrete, 2-in. slump. Each value is the average of five tests made on different days. Concrete was mixed in a 3-S tilting drum mixer. The mixer was buttered by mixing and discarding one batch before each day's use. Values shown are relative to 1½-in. aggregate after standard curing to 28 days.

Maximum size of aggregate, in.	Flexural strength 12 x 8 x 30-in. beams		Compressive strength 6 x 12-in. cylinders	
	Aggregate A	Aggregate B	Aggregate A	Aggregate B
2½	103	102	—	—
1½	100	100	100	100
¾	86	96	84	81

TABLE K

6-sack concrete, 2½-in. slump. Aggregate A. Each value is the average of nine specimens, each from a separate batch. Concrete was mixed in an open tub mixer. Values shown are relative to 1½-in. aggregate.

Maximum size of aggregate, in.	Flexural strength 5 x 4 x 18-in. beams in fog room 28 days then in air 28 days*	Compressive strength 6 x 12-in. cylinders standard cured to 28 days
1½	100	100
¾	96	90
⅜	86	69

*These specimens were used first for drying shrinkage tests as reported in Table M.

TABLE L

Comparison of compressive strength of 5-in. cores cut from pavement and 6 x 12-in. cylinders molded from concrete as placed. Specimens were moist cured for 7 days. Values are relative to those produced by 1½-in. aggregate.

Type of concrete	Compressive strength	
	Cores	Cylinders
5-sacks, 1½-in. aggregate	100	100
5-sacks, ¾-in. aggregate	82	89
6-sacks, 1½-in. aggregate	100	100
6-sacks, ¾-in. aggregate	89	102

TABLE M

Drying shrinkage of 5 x 4 x 18-in. prisms with reference points. Length changes were measured with a freely suspended horizontal comparator. Standard cured for 28 days, then dried at 73 F and 50 percent relative humidity. Each value is the average of 9 specimens from separate batches.

Maximum size of aggregate, in.	Shrinkage after drying for period indicated			
	14 days		28 days	
	Percent	Relative	Percent	Relative
1½	0.0121	100	0.0166	100
¾	0.0146	121	0.0217	131
⅜	0.0157	130	0.0262	158

On another project constructed between side forms with consolidation by conventional reciprocating screeds, 1½-in. and ¾-in. aggregates were used, each in 5- and 6-sack concrete. During construction, 6 x 12-in. test cylinders were molded by trained laboratory personnel. The cylinders were cured adjacent to the pavement until cores were cut. Subsequently the cores and cylinders were moist cured similarly until tested for compressive strength at the age of 7 days. Test results, referred to the strength of concrete containing 1½-in. aggregate in each case, are shown in Table L. It will be noted that the reduction in strength as a result of using ¾-in. aggregate was greater in the pavement cores than in the test cylinders.

The results of these tests bring out clearly that standard test specimens reflected only partially the effect of size of aggregate that was obtained in pavement construction.

The authors' results for the effect of aggregate size on drying shrinkage are similar in trend to those determined by the California Division of Highways, the data of which are given in Table M. The authors' comment that difference in drying shrinkage of 0.005 percentage point would seem to be of little or no significance warrants discussion.

On a relative basis, the drying shrinkage produced by ¾-in. aggregate was 115 percent of that of the 1½-in. aggregate. The ⅜-in. aggregate produced drying shrinkage of about 140 percent of the 2½-in. aggregate. The data given in Table M indicate that the differentials should increase with longer periods of drying. The results obtained with California aggregates indicate possibly greater differentials, although the results cannot be compared directly because of differences in test conditions. The results suggest, however, that differences found by the authors may be greater when other aggregates are used.

The writer believes that differences in drying shrinkage, even though of relatively small absolute magnitude, are of real significance in the performance of pavements and structures. Hveem[34] and Hveem and Tremper[35] reported that pavement slabs warp upward as a result of differential drying between top and bottom. In a warped condition, the slabs deflected more under load. The greater the drying shrinkage, the greater is the upward warping and the greater is the deflection under load.

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By LEWIS H. TUTHILL*

The authors have presented a most tempting prospect. We recall many times that aggregate large enough to be seen in the mix has been protested—and had to be defended, we thought for the sake of better concrete and economy. We are reminded of the many areas of the country where larger gravel is increasingly absent from natural deposits, and has to be imported at considerable expense. Recently we have seen the separation that occurs when truck mixers discharge concrete containing the larger gravels, separation which necessitates special vigilance throughout handling and placing operations. When we encounter these problems, it would be so much easier if we could accept the authors' alluring conclusions and let the contractors use whatever small gravel they happened to have at hand and prefer to use.

However, before we can fully believe this utopia has arrived, we shall have to see a substantial confirmation of their conclusions in the results of a considerable amount of contemporary research on this subject.

In the meantime, it may be worthwhile to look at some data that is contemporary with the authors', and to look again at some that is older. All of these data fall considerably short of confirming that ¾-in. aggregate can be expected to produce about the same strength as properly proportioned concrete with larger aggregate at the same cement content and slump, particularly at the more common structural strength levels for "3000 lb" concrete.

When the authors first announced their attractive conclusions, a limited program of tests was undertaken. Results in Fig. O and P are averages of 12 test cylinders made in four rounds of three each for each value shown. Tests were made with each maximum size mix in 8 x 16-in. and 6 x 12-in. cylinders, and also in cylinders of a diameter related to the maximum size of aggregate, viz.: 2 x 4-in. for ⅜-in.; 3 x 6-in. for ¾-in.; 6 x 12-in. for 1½-in.; and 8 x 16-in. for 2½-in. All mixes contained the same amount of cement per cubic yard and had the same slump. Air was omitted for easier laboratory control and because many of the authors' tests contained no air.

It should be noted that the percent of sand in the mixes shown in Fig. O closely follow amounts for workable but not oversanded mixes recommended in the Bureau of Reclamation *Concrete Manual*, Sixth Edition,[16] Table 14, p. 120, with minor modifications appropriate for the aggregate used. Contrary to this time-tried approach for selection of proper sand content, it will be noted in the authors' Table 2 that the percent of sand in their mixes was remarkably uniform for the ¾-, 1½- and 2½-in. aggregates and much too low for the ⅜-in. aggre-

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gate. Amounts of sand chosen for the 1½-in. aggregate were reasonable, but the low amounts used with smaller aggregate, inadequate as they must have been from the standpoint of job workability requirements, required less than usual amounts of mixing water with resultant lower W/C and higher strengths than would ordinarily be obtained. Conversely, somewhat more than the correct amount of sand for 1½-in.

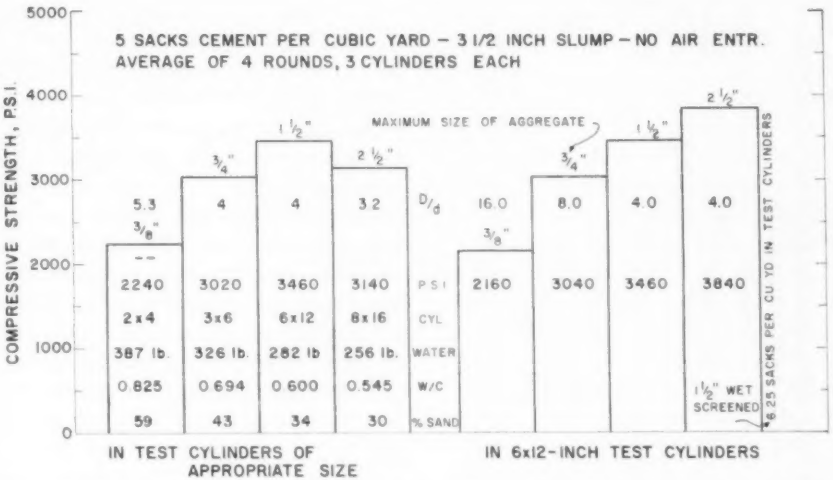
5 SACKS CEMENT PER CUBIC YARD – 3 1/2 INCH SLUMP – NO AIR ENTR.
AVERAGE OF 4 ROUNDS, 3 CYLINDERS EACH
COMPRESSIVE STRENGTH, P.S.I.
5000
4000
3000
2000
1000
0
MAXIMUM SIZE OF AGGREGATE
3/8" 3/4" 1 1/2" 2 1/2" 3/4" 1 1/2" 2 1/2"
D/d 5.3 4 4 3.2 16.0 8.0 4.0 4.0
PSI 2240 3020 3460 3140 2160 3040 3460 3840
CYL 2x4 3x6 6x12 8x16
WATER 387 lb. 326 lb. 282 lb 256 lb.
W/C 0.825 0.694 0.600 0.545
% SAND 59 43 34 30
1 1/2" WET SCREENED
6.25 SACKS PER CU YD IN TEST CYLINDERS
IN TEST CYLINDERS OF APPROPRIATE SIZE
IN 6x12-INCH TEST CYLINDERS

Fig. O—Effect on 28-day strength of maximum size of aggregate in concrete of same cement content and slump

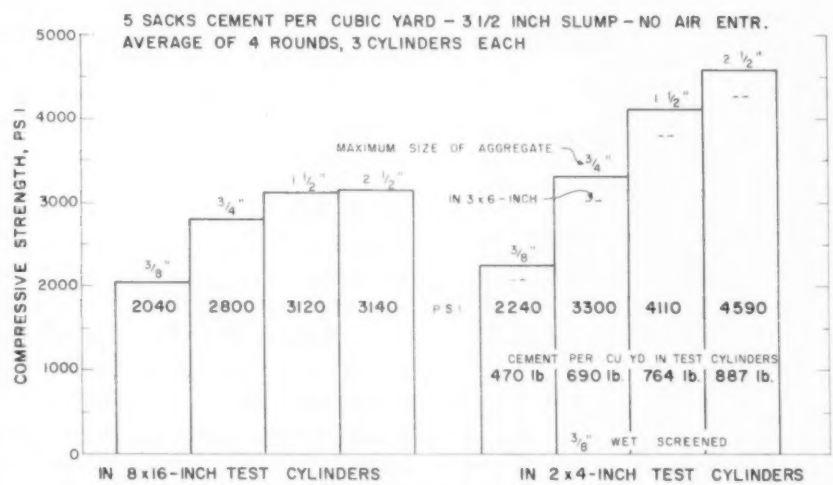
5 SACKS CEMENT PER CUBIC YARD – 3 1/2 INCH SLUMP – NO AIR ENTR.
AVERAGE OF 4 ROUNDS, 3 CYLINDERS EACH
COMPRESSIVE STRENGTH, P.S.I.
5000
4000
3000
2000
1000
0
MAXIMUM SIZE OF AGGREGATE
IN 3x6-INCH
3/8" 3/4" 1 1/2" 2 1/2" 3/8" 3/4" 1 1/2" 2 1/2"
PSI 2040 2800 3120 3140 2240 3300 4110 4590
CEMENT PER CU YD IN TEST CYLINDERS
470 lb 690 lb. 764 lb. 887 lb.
3/8" WET SCREENED
IN 8x16-INCH TEST CYLINDERS
IN 2x4-INCH TEST CYLINDERS

Fig. P—Effect on 28-day strength of maximum size of aggregate in concrete of same cement content and slump

TABLE N — EFFECT OF MAXIMUM SIZE OF AGGREGATE ON CONCRETE CEMENT CONTENT, W/C, AND STRENGTH

Maximum size aggregate,	Table 14,* water required, lb per cu yd	Fig. 28,* W/C	Indicated cement per cu yd, bbl	Fig. 28,* strength, psi per bbl	(1) Indicated strength, psi	Assume same strength with each size aggregate	Cement content for same strength as with ¾-in. maximum aggregate, bbl per cu yd	(2) W/C at cement content required for same strength	(3) Strength† for same cement content and slump	
									W/C	psi
	(Based on water required for non-air-entrained concrete)								(Assuming 1.30 bbl per cu yd)	
¾	316	0.54	1.56	2460	3840	3840 (÷ 2460 =)	1.56	0.54	0.63	3200
1½	280	0.54	1.38	2665	3680	3840 (÷ 2665 =)	1.44	0.52	0.57	3460
3	242	0.54	1.19	2965	3530	3840 (÷ 2965 =)	1.30	0.50	0.50	3850
6	210	0.54	1.03	3295	3400	3840 (÷ 3295 =)	1.165	0.48	0.43	4280

*Table 14 and Fig. 28, Bureau of Reclamation *Concrete Manual*. Data in Fig. 28 is for non-air-entrained concrete.
†Contrary to this, the authors conclude that at same cement content and slump, better strength is obtained with smaller maximum size aggregate.

aggregate was grossly excessive for 2½-in. aggregate and, by requiring more mixing water, could be expected to explain reduced resulting strength. Such abnormal water contents would also tend to explain the authors' doubtful conclusion that "differences in drying shrinkage were probably so small as to be of little practical significance in most concrete."

Results of our tests of properly graded mixes are self-evident on the bar graphs. In *no case* was the concrete with ¾-in. or smaller aggregate superior in strength to concrete with 1½-in. aggregate. The test of wet-screened 2½-in. aggregate concrete was superior in strength to that of all other smaller sizes, and only with the full mix in the relatively restricted 8 x 16-in. cylinders did the 2½-in. aggregate exhibit somewhat less strength than 1½-in. aggregate concrete.

In addition to these tests a study was made of Fig. 28 in the USBR *Concrete Manual*,[16] here shown as Fig. Q. This curve shows the strength efficiency of cement in psi per bbl of cement per cu yd for concrete with various maximum sizes of aggregate, as determined from the average strengths of many tests in test cylinders of appropriate diameter for the size of aggregate. The relevance of this curve was questioned with the statement that it was based on "completely idealized relationships." This term is misleading and incorrect in its implication that in drawing the curve the data were

not altogether heeded, perhaps to obtain a curve that was consistent with preconceived or commonly held beliefs as to what these relationships should be. Certainly the actual points representing the average strength of concrete with each size of aggregate would not have been shown had they not been authentic.

The indicated strength per barrel of cement per cubic yard for concrete of each maximum size of aggregate, is shown in the left group on the bar graph of Fig. R. It is readily evident that with this or a different cement content but the same for all sizes of aggregate, strength is not the same but definitely increases for each increase in maximum size of aggregate. It is clear that ¾-in. aggregate did not produce an equal or better strength than larger aggregates.

However, in Table N, further analysis of these data indicates, for the same slump, that (1) for the same W/C, strength decreases as maximum size of aggregate increases; (2) at the cement content required for the same strength, the W/C decreases as the maximum size of aggregate increases; (3) at the same cement content, strength increases and W/C decreases as the maximum size of aggregate increases in the concrete.

Where durability resistance to freezing is an important consideration, approximately one-third of this resistance can be lost by using a ¾-in. aggregate mix instead of a 1½-in. aggregate mix at the same

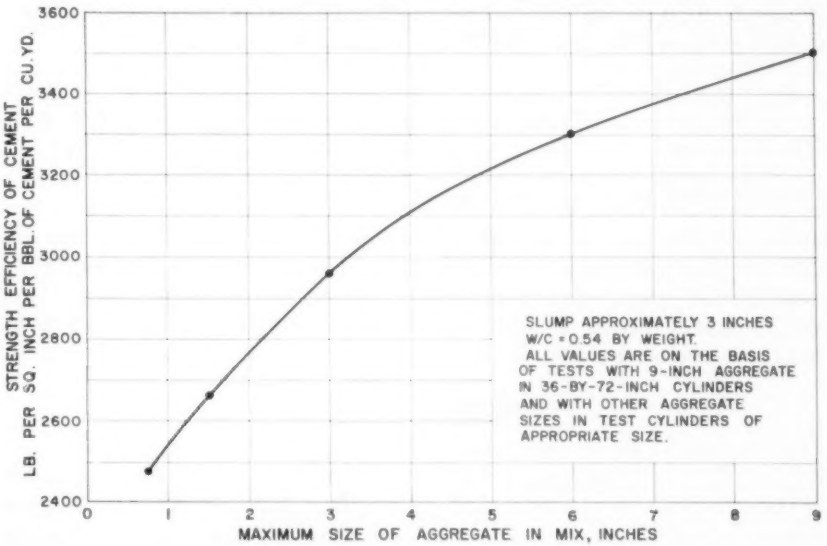

Fig. Q—Strength efficiency of cement for concrete made with various maximum size of aggregate. More strength per barrel of cement is obtained in mixes having large coarse aggregate. Plotted from values obtained by dividing the average strengths at 28, 90, and 365 days by the cement for each maximum size (USBR Concrete Manual, Fig. 28)

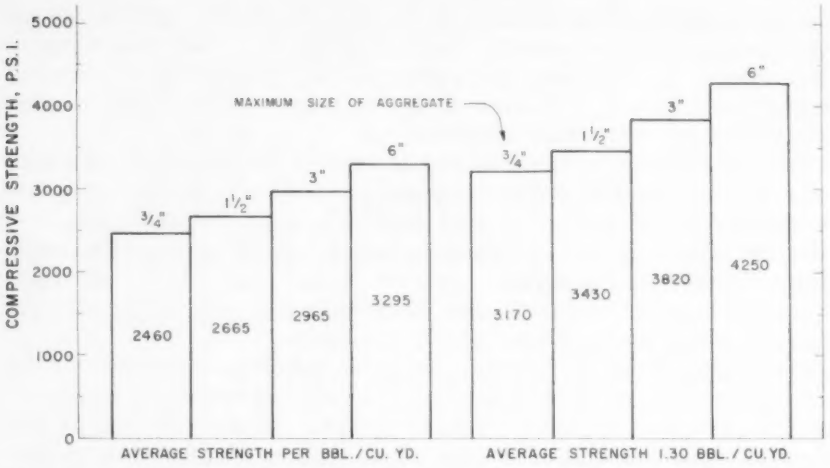

Fig. R—Effect on strength of maximum size of aggregate in concrete of same cement content and slump (Tests in Fig. 28 of USBR Concrete Manual)

cement content and slump. This is because of the greater water content and higher W/C in the ¾-in. aggregate concrete. Again referring to the *Concrete Manual*,[16] Table 14 and Fig. 16, and assuming the proper W/C of about 0.50 for air-entrained weather resistant concrete of 1½-in. maximum size, the following comparison emerges:

Maximum size, in.	Water per cu yd, lb*	Cement per cu yd, lb	W/C	Durability factor*
1½	245	494	0.496	10.1
¾	283	494	0.574	6.7

*Table 14 and Fig. 16, Bureau of Reclamation *Concrete Manual*.[16]

The durability factor, according to this data is decreased one-third (10.1 to 6.7) by using a ¾-in. aggregate mix instead in a 1½-in. aggregate mix. It might also be said that the durability factor might well be increased by half (6.7 to 10.1) if 1½-in. was used instead of ¾-in. maximum sized aggregate.

By PAUL S. WRIGHT*

The communicant has the highest respect for the writers of this paper and for Professor Gilkey. I have had the pleasure of meeting these gentlemen while associated with Committee C-9, ASTM, and it goes without saying that they are true research scientists in the field of concrete technology and each is outstanding in his individual contributions of technical advancement in this field.

No real conflict with Abrams' W/C axioms appears to me in a careful

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perusal of these data. We may, I think, still consider: (a) That for a given cement factor and a given grading, any reduction in the water content (consistency or slump) will result in an increase in strength at all periodic intervals. (b) That any reduction in maximum aggregate size will result in greately increased surface area, necessitating a commensurate increase in water content; and a commensurate increase in cement factor to maintain the design W/C ratio.

It has been the observation of the City of Los Angeles, in the construction of perhaps $150,000,000 of reinforced concrete work since 1945, that, given a constant cement factor (6.4 sacks per cu yd) and a constant slump requirement, a change from 1-in. maximum aggregate grading to 1½-in. maximum aggregate grading has enabled such reductions in water content per cubic yard as to result in strength increases of 500 to 1000 psi at 28 days.

This has been observed to be true in all work done under careful inspection and especially graphic under conditions of excellent inspection at reinforced concrete pipe yards.

It would be interesting to know if the test specimens underwent any type of revibrating at or near the initial set time of the concrete. This is important for the following reasons.

Placed concrete is subject, prior to initial set, to upward migrating water. Often migratory water channels terminate at the lower side of larger aggregate particles, water collecting and staying at those points. If revibration is performed, these water deposits are "shaken out" from under the aggregate particles, resulting in increased strength, and other qualities of the concrete. Inversely, if revibration is not done, it is my opinion that the water migration deposits under larger aggregate particles could cause coarser aggregate mixtures to appear weaker in flexural and compressive strengths than the real inherent qualities of such coarser mixtures; i.e., exhibit the apparent differences in these qualities shown in the paper for various maximum sizes of aggregates.

For whatever value this comment may have, it is interesting to note that in asphalt concrete the mixtures gain, progressively, in stability, beam strength, and other desirable qualities, as the gradings of the mixtures are coarsened (the maximum size of aggregate is increased).

By N. G. ZOLDNERS*

The authors point out that the universally recognized law of water-cement ratio may not be true if different maximum sizes of coarse aggregate are used. Results of the investigation conducted by the authors indicate that an increase in the maximum size of coarse aggregate does not necessarily result in stronger concrete.

The writer fully supports the authors' statement that in richer mixes

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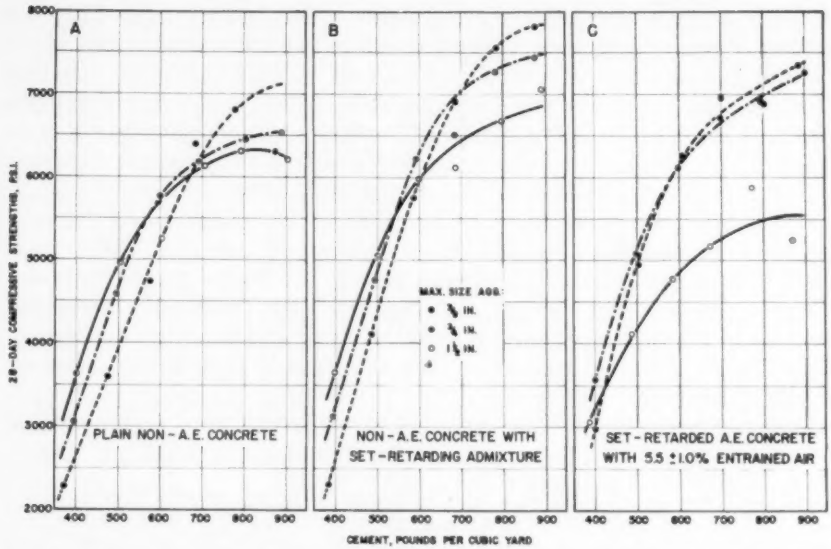

Fig. S—Effect of cement content on compressive strength for various maximum sizes of aggregate in different types of concrete

the strength of concrete decreases if larger sizes of coarse aggregate are used. Determination of the optimum size of coarse aggregate for different concrete strength levels has a practical value for concrete producers. However, the optimum size may vary when different types of cement, aggregate, admixture, and amount of entrained air are used.

In the opinion of the writer the publication of additional experimental results from tests conducted elsewhere on different types of concrete would further the understanding of the various factors involved in concrete strength development.

A recently completed test series by the writer substantially confirms the results published by the authors. These tests were conducted on five strength levels with cement content varying from 400 to 900 lb per cu yd. A set-retarding admixture was used in both non-air-entrained and air-entrained concrete series. Also a series of plain concrete mixes were tested for purposes of comparison. The coarse aggregate used was a subangular, igneous gravel from an operating pit of a glacial-fluvial deposit near Ottawa, Ont., Canada. The grading of the ⅜, ¾, and 1½ in. maximum size stone was similar to that grading adopted by the authors. The results obtained in these tests are presented in the form of graphs similar to those used by the authors. However, for further clarification, the vertical scale showing compressive strength has been enlarged twofold.

Fig. S shows the effect of added cement on the 28-day compressive strength for three maximum sizes (⅜, ¾, and 1½ in.) of coarse aggre-

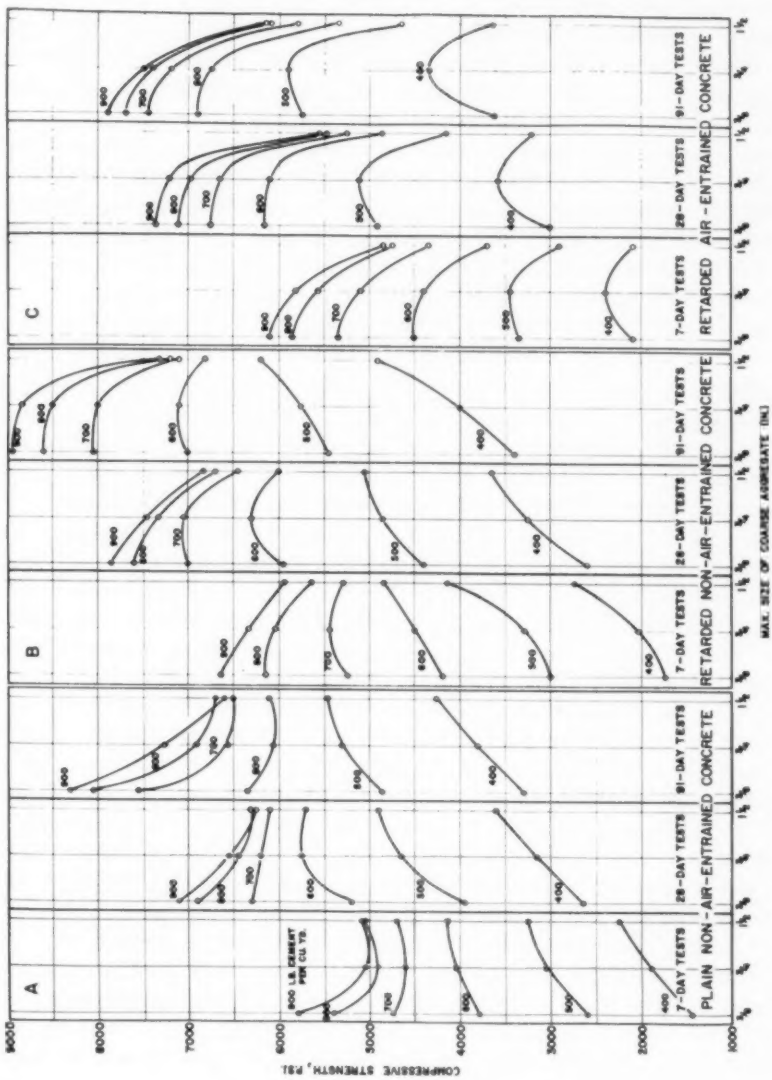

Fig. T—Effect of size of coarse aggregate on compressive strength in different types of concrete

gate in three types of concrete. The slump of all test mixes was maintained at approximately 4 in.

Graphs A and B in Fig. S show that in non-air-entrained concrete the 1½-in. size aggregate is beneficial in leaner mixes, whereas in richer mixes, with 700 lb of cement per cu yd and over, the ⅜-in. size aggregate produces higher strength. Graph C shows that air entrainment substantially reduces the strength of concrete made with 1½-in. size aggregate, but has little effect on concrete made with the smaller stone.

These graphs show clearly that, in concrete with cement content exceeding 600 or 700 lb per cu yd, higher compressive strength can be produced with the smaller stone (i.e., ⅜-in. maximum size aggregate).

Graphs in Fig. T, plotted from data derived from the curves in Fig. S, show the effect of maximum size of aggregate on compressive strength of concrete with varying cement content at 7, 28, and 91 days' age.

AUTHORS' CLOSURE

The authors are gratified by the interest displayed in their paper, as evidenced by the number of discussions. Time and a proper consideration of *Proceedings* space will not permit detailed comment on each. However, all discussors may be assured that their searching and constructive comments have been studied and are appreciated.

Messrs. Zoldners, Mitchell, Frazier, Howard, Mead, and Macnaughton have presented data which are generally corroborative of our own. The authors are in full agreement with Mr. Zoldners and others that the optimum maximum size varies with a number of factors such as richness of mix, type of aggregate, amount of entrained air and, no doubt, others. We note the markedly low strengths of the 1½-in. maximum size, set-retarded air-entrained concrete shown in Mr. Zoldner's Fig. S. It would be of interest to know just what was the cause of this seemingly abrupt departure from the pattern established by the other data.

Mr. Mitchell presents data both pro and con. Tests with a quartz gravel, in concrete containing 6 sacks of cement per cu yd, gave results similar to our own. Companion tests with a crushed granite support the conventional size-strength relationship. The authors suggest the possibility that the use of a higher cement factor than 6 sacks with the granite might have shown less advantage for the larger sizes.

Mr. Frazier's results parallel the authors' very closely. It should be of interest to note that the gravel used by him is from the same source as that used by us. Because of its uniformly high quality, it is the "laboratory standard" used in the NSGA-NRMCA Joint Research Laboratory at the University of Maryland, as well as by other laboratories. It should also be noted that Mr. Frazier reports companion tests with crushed stone which produced relationships in good agreement with those for his tests with gravel.

Mr. Howard's succinct contribution is appreciated. Mr. Mead's presentation of data in terms of "strength efficiency" (psi per pound of cement per cubic yard of concrete) is most interesting and enlightening. We have examined our own data by the same procedure and find them to fall in a similar pattern. The concept that each maximum size of aggregate has an optimum cement content so far as strength efficiency is concerned deserves further development. The Bureau of Reclamation has shown a similar efficiency measure to increase sharply with maximum size, but it is not clear that the values are for optimum cement content for each size.

Mr. Macnaughton not only presents corroborative results, but he points to what must be a major factor in the relationships developed—internal stresses due to shrinkage. We believe that his data relating flexural to compressive strength should be considered as offering support to that thesis. The authors are in full agreement with Mr. Macnaughton that much additional research should be conducted to discover what basic principles are involved.

Mr. Spindel has read into the paper more than the authors have said. His quotation, "the strength of the concrete was progressively reduced as maximum size was increased above ¾ in. or in most cases above ⅜ in.," is out of context. He should have included the forepart of the sentence which reads, *"In all air-entrained concrete and all but the leanest non-air-entrained concrete,"* (the italics have been added). We also said, "It seems probable that the relationship between the strength and maximum size will vary depending upon the characteristics of the aggregate." Some preliminary results of further investigations along these lines are included herein.

Mr. Spindel's comments on shrinkage during hardening, with the consequent development of internal stresses differing with different sizes of coarse aggregate, is of interest. The authors agree that this very likely is one of the factors involved. Consideration also should be given to the effect of the greater surface area available for bond and the greater cross-sectional area to resist shear stresses for the smaller sizes than the larger. Mr. Spindel's comments on some of the earlier developments in concrete technology, and particularly on the fineness modulus, are of special interest to the senior author who worked with Abrams during the development of his theories of proportioning concrete.

Messrs. Alexander and Wardlaw refer to their earlier research on cement-aggregate bond. The authors read Mr. Alexander's paper in the 1959 ACI *Proceedings* with interest, at the time of its publication. That their work in this field has a direct bearing on the current discussions, there can be no doubt. However, as they recognize, bond strength alone does not offer sufficient explanation.

Messrs. Alexander and Wardlaw raise an interesting question as to just how concrete should be visualized. Is it a two-phase system of

mortar and coarse aggregate or of cement paste and total aggregate? They suggest, correctly we believe, that neither is correct and discuss some of the limitations.

At the risk of over-simplification, the authors suggest that concrete may be considered as a two-phase system, with one phase being aggregate larger than some size and the other the mixture of water, cement, and aggregate smaller than that size. It seems reasonable that the dividing size will change with different materials, proportions, and other factors. It is suggested that for all practical purposes a relationship so qualitative may be viewed in terms of conventional mortar as one phase and conventional coarse aggregate as the other. The fact that the fine and coarse aggregate frequently differ substantially, mineralogically and otherwise, makes a more refined concept all the more difficult to visualize.

Messrs. Ali and Lingam "suggest a possible explanation for the observed reduction in concrete strength" based on: (1) strength of cement paste; (2) bond strength between cement paste and aggregate; and (3) the geometry of the aggregate binder system. Except for details of expression, others participating in this discussion, including the authors, appear to agree with the principles of this concept.

Mr. Holleb expresses the opinion that discontinuous gradings are preferable to continuous ones and that different gradings of fine aggregate should be used with different sizes of coarse aggregate. He suggests that, since the authors did not follow these precepts, their results are open to question. We will not at this time enter into a discussion of the relative merits of continuous versus discontinuous gradings or of optimum gradings of fine aggregate for different sizes of coarse aggregate. We will say only that in this country conventional practice is to use essentially continuous gradings and the same grading of fine aggregate for different coarse aggregate. Our tests had the objective of furnishing information pertinent to conventional practices with respect to aggregate grading—and, it should be added, practices which the authors believe to be based on sound engineering principles.

Mr. Thaulow's comment that our tests "will have future significance for concrete technology" is appreciated. His suggestion that the geometry of the test specimens may have affected the results deserves study. He points out that the cube-cylinder ratio is so variable that there is danger of drawing different conclusions depending on the shape of the test specimens. The authors agree emphatically that there should be standardization, preferably, in their opinion, by acceptance of the 6 x 12-in. cylinder on which such a vast amount of data is based. We have studied Gyengo's paper, to which Mr. Thaulow refers, and do not find data which we interpret as applicable to the current discussion, since neither absolute strengths nor values proportional to them are given.

The authors have read with interest Mr. Thaulow's comments on the "split-cylinder tests" which we reported.

The authors thank Mr. Wright for his friendly discussion. We agree with him that our findings present no real conflict with the water-cement ratio law. They do, however, add another limitation to the many already recognized. As to the method of molding our specimens, they were not vibrated. They were made in strict accord with standard ASTM procedures.

Mr. Tremper presents data which do not confirm the authors' findings. However, the degree of lack of support varies with the different groups of data. In Table J, flexural strengths for two aggregates in 5-sack concrete using 2½-in. aggregate are shown to be only 103 and 102 percent of those for 1½-in. aggregate. Also, for one of these, the flexural strength for the ¾-in. aggregate was as much as 96 percent of that for the 1½-in. material. The smaller size did not fare so well in compression. In 6-sack concrete (Table K), the flexural strength for ¾-in. aggregate was again 96 percent of that for 1½-in.; and in Table L compressive strength

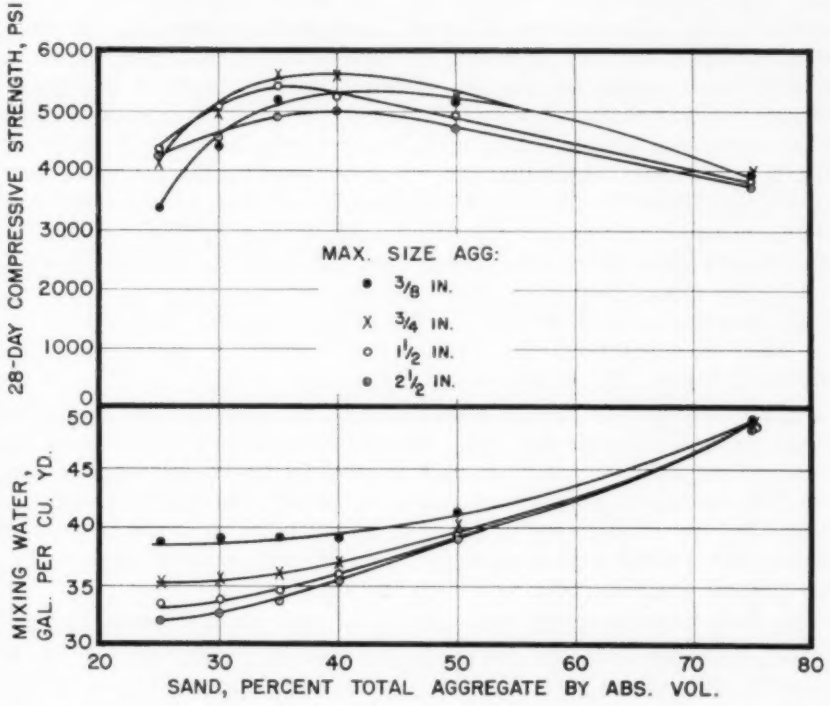

Fig. U—Effects of aggregate size on water requirement and strength of concrete (Series J-112)

of molded cylinders of 6-sack concrete for ¾-in. aggregate was 102 percent of that for 1½-in.

Admittedly, in the preceding we have selected the favorable data, but they involve nine of the 24 values in Tables J, K, and L.

A feature of Mr. Tremper's findings on which further data should be of great interest is that which indicates the smaller aggregates to show to greater disadvantage in cores drilled from the pavement than in molded specimens. Does this suggest that the methods used afforded better compaction for the larger aggregate than for the smaller? If so, was this due to size *per se*, or to other factors?

Mr. Tremper's suggestion that coarse aggregate characteristics influenced the results is borne out by further tests made by us, preliminary results of which are reported herein. We are gratified that the drying shrinkage tests reported by Mr. Tremper appear to be in line with our own. The authors' data for shrinkage after 1 year are also included later in this closure.

We are appreciative of Mr. Hawkins' comments on concrete design methods published by the senior author some years ago. Both he and Mr. Tuthill question the ratios of fine to coarse aggregate used in the authors' tests. As stated in the paper, these were selected on the basis of comprehensive tests of 6-sack non-air-entrained concrete, reported in a previous paper, the results of which are summarized in Fig. U. As may be seen from Table 2 of the paper, the fineness moduli of combined aggregate progress with size in an orderly fashion. A study of Fig. U will make it clear that comparisons based on the same percentages of sand used by Mr. Hawkins would have given substantially the same results. Mr. Hawkins' tabulated data provide further support for the authors' observation that larger sizes produce lower strengths than smaller ones at a given water ratio. His 6-sack concretes with ¾-in. and ⅜-in. maxima gave substantially the same strengths, particularly at the older ages, although the smaller size required a water ratio substantially higher (7.1 as opposed to 5.8 gal. per sack).

In the case of Mr. Tuthill, comparison of his sand percentages with those in Fig. U suggests that using his values would not have altered the trends of the authors' results, except possibly for the ⅜-in. size. For that size, the discussor's proportions appear to be over-sanded by usual criteria. No real assessment of the agreement of the percentage of sand used by Mr. Tuthill with usual criteria can be made without sieve analyses, specific gravities, and unit weights.

The only new strength data developed for the discussion by Mr. Tuthill were for concrete containing 5 sacks of cement per cu yd of concrete. It appears that the cement was fairly low testing since his 5-sack concrete gave strengths comparable with ours containing only 4 sacks.

The only data permitting of direct comparisons for all sizes are shown in bar graphs for the 8 x 16-in. cylinders. Comparison with corresponding data in Table 3 of the paper (for the 4-sack concrete, with about the same strength) will show remarkably similar indications. The strength range chosen by Mr. Tuthill was at this moderately low level where the authors noted that large sizes fare best. Here the 1½- and 2½-in. aggregates gave substantially the same results.

It is suggested that the use by Mr. Tuthill of a higher strength concrete or a higher cement factor would have shown the smaller sizes to better

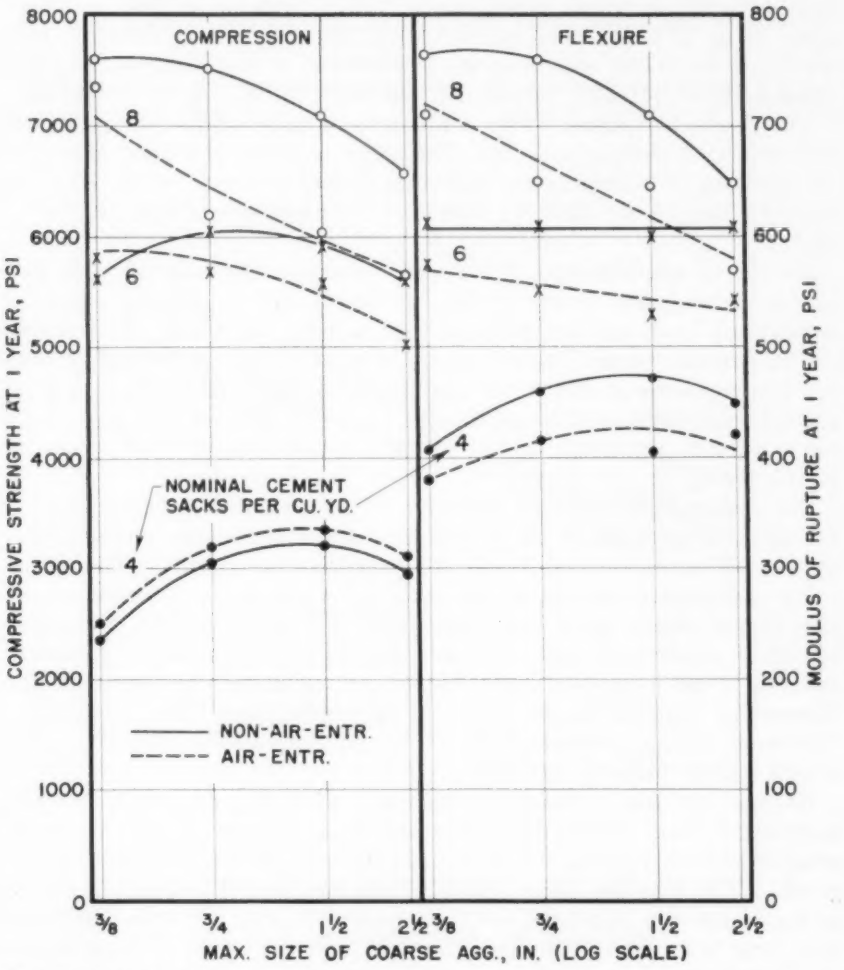

Fig. V—Effect of size of coarse aggregate on 1-year strength of concrete (Series 173)

advantage. Tests of wet-screened concrete are, of course, not valid in a comparison such as this. Not only is the "size effect" eliminated by discarding the larger sizes, but there is a large range in cement content of test specimens, increasing from 470 lb to 887 lb per cu yd as maximum size increased.

Analysis of Mr. Tuthill's Fig. Q lends support to the conclusion that large sizes develop lower strengths at the same water ratio than small ones. Applying data from Fig. 27 of the Bureau of Reclamation *Concrete Manual,* we find that, for the constant water ratio of 0.54 illustrated in Fig. Q (Bureau of Reclamation *Concrete Manual* Fig. 28), compressive strength decreased from about 3800 to 3400 psi as size increased from ¾ in. to 6 in. Here again, we are dealing with moderately lean mixtures where the combination of conditions is such that, at the same cement factor, strength would increase with increasing maximum size.

The research reported by the authors has yielded additional data since our report of about a year ago. The original paper contained strengths for ages up to 3 months and shrinkage measurements for 6 weeks of drying. The 1-year data are now available and are shown in Fig. V and W.

So far as relationships of strength to maximum size of aggregate are concerned, the 1-year results shown in Fig. V do not alter the indications from earlier test ages reported in the paper. The shapes of the strength versus size curves are similar to the corresponding ones for 3-month tests in Fig. 1 through 4 of the paper. Strength is highest for the intermediate or small sizes in all cases. The smaller sizes show up to greater advantage as the concrete becomes richer and when air is entrained.

The one marked contrast between 1-year and earlier results occurs for flexural strength of the 8-sack mixtures. There, the strengths for all sizes of aggregate were lower at 1 year than at 3 months. The reductions were greater for the larger sizes. As a result, the strength versus size curves which were essentially flat at 3 months show a marked reduction in strength with increased size in the 1-year tests. It is conceivable (and consistent with hypotheses advanced by some of the discussors) that the larger pieces of aggregate cause greater restraint to normal volume changes of the mortar, thus producing internal strains which reduce flexural strength.

Drying shrinkage measurements, taken after 1 year at 100 F, when equilibrium had essentially been reached, are shown in Fig. W. Indicated effects of aggregate size are similar to those for 6 weeks' drying given in Fig. 9 of the paper. Both weight loss and shrinkage decreased as maximum size increased, with greatest difference occurring between ⅜ in. and ¾ in. The shrinkage difference over the range of usual aggregates, from ¾ in. to 2½ in. maximum size, amounted to less than 0.01 percentage point.

While the leanest concrete (4 sacks per cu yd) developed more shrinkage at 6 weeks than the two richer mixes, it developed the least shrinkage after 1 year. It appears that the richer mixes lost water and shrank more reluctantly at first, but over a longer period, than the lean mix.

There was no difference in total shrinkage between air-entrained and non-air-entrained concretes, although the former lost slightly less weight during drying. Total shrinkage was about the same for the 6- and 8-sack mixes but, as already mentioned, slightly less for the 4-sack mix.

Additional studies of maximum size are being made with aggregates from over 50 sources throughout the country and including one from Canada and one from England. Preliminary data for the first 12 sources

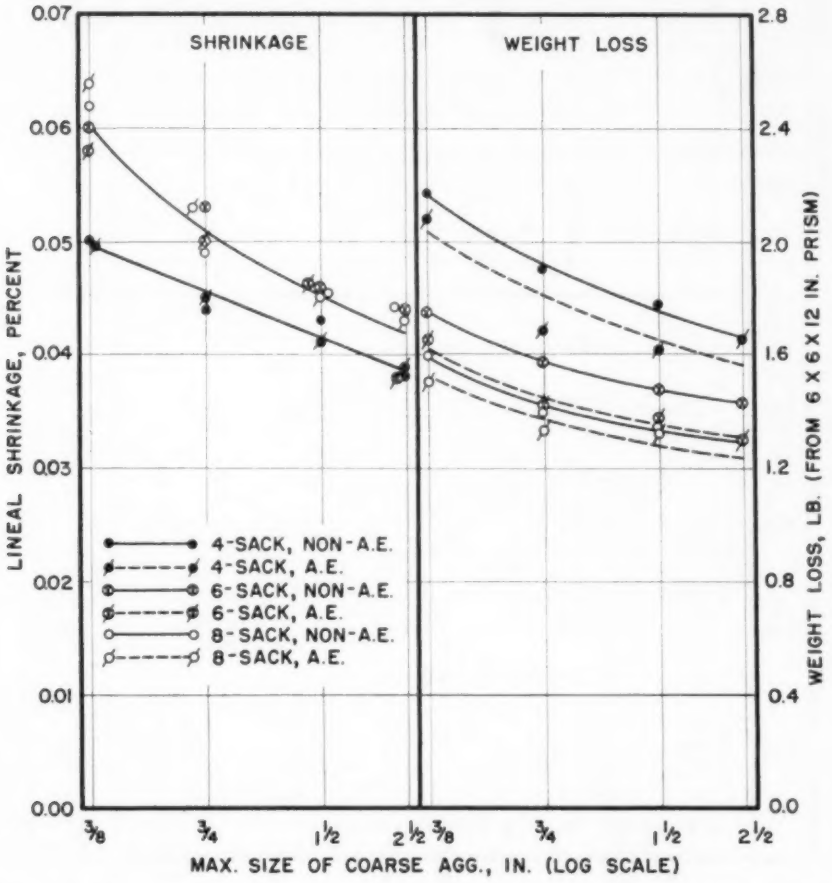

Fig. W—Drying shrinkage and weight loss of concrete in 1 year at 100 F (Series 173)

tested are shown in Fig. X. They present 28-day strengths and water requirements for ¾-in. and 1½-in. maximum sizes in 5-sack and 7-sack mixtures. The concretes were non-air-entrained with aggregate proportions selected by conventional criteria of fineness modulus and b/b_o. Sand contents ranged from about 37 to 44 percent of total aggregate for

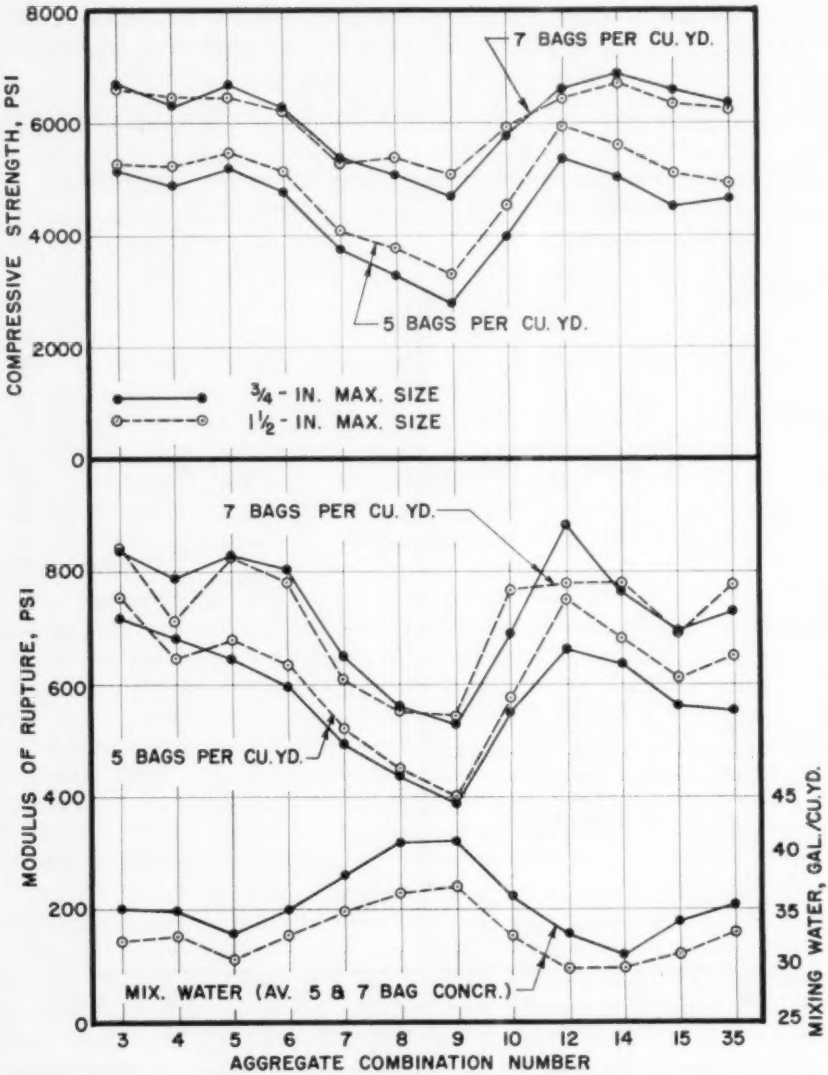

Fig. X—Effect of maximum size of aggregate on strength of concrete (Series 178)

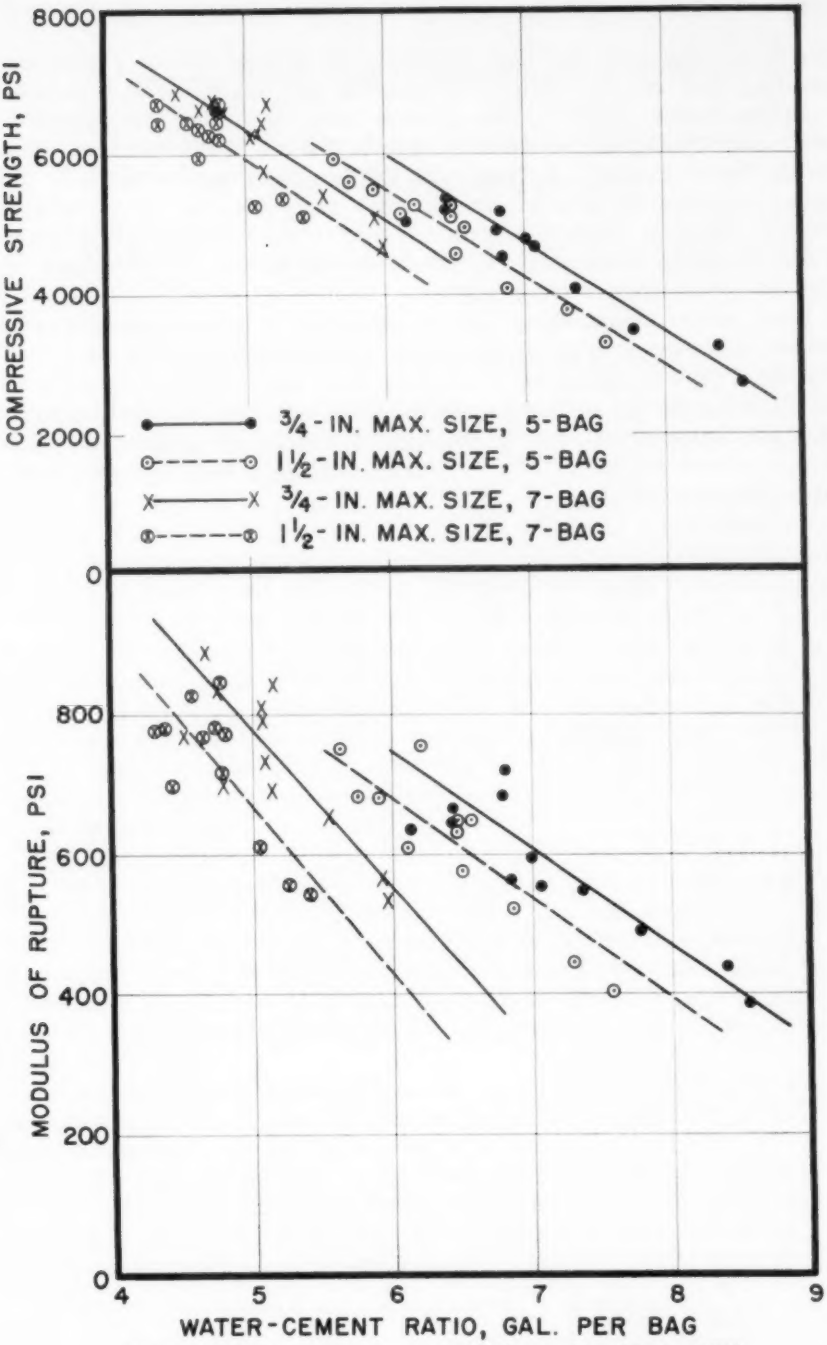

Fig. Y—Water-ratio versus strength relationships (Series 178)

the ¾-in. maximum size and from 31 to 37 percent for the 1½-in., depending upon cement factor, sand grading, and aggregate void content.

In the 5-sack concrete, strengths in both flexure and compression were slightly higher for the 1½-in. than for the ¾-in. maximum size. In the 7-sack concrete, strengths for the two sizes were essentially the same, favoring the smaller size in some cases and the larger size in others. The most notable indication is that strength differences between sizes for either cement factor were inconsequential in comparison to the differences between aggregates.

Basic effects of aggregate size are indicated by the water-ratio relationships shown in Fig. Y. Although there is considerable scattering for the different aggregates, there is a clear separation in water-ratio-strength curves not only between the different sizes but also between the different cement factors. At a given water ratio, higher strength was produced by ¾-in. than by 1½-in. aggregate and by 5-sack than by 7-sack concrete.

Discussion of a paper by Joseph Di Stasio, Sr. and M. P. van Buren:

Transfer of Bending Moment Between Flat Plate Floor and Column*

By F. D. BERESFORD and FRANK A. BLAKEY, ROBERT R. KREPS and RAYMOND C. REESE, and AUTHORS

By F. D. BERESFORD and FRANK A. BLAKEY†

The effect of the flexural restraint by a column on a flat plate is of particular interest in Australia, where this type of structure is being used in rapidly increasing numbers.

Although we have not made any tests in the form suggested by the authors, in the course of tests on an experimental flat plate structure[1,2] and on an actual flat plate building we have obtained results relevant to the problem discussed by them. Perhaps the most interesting of these results concerns torsion T. Both of the structures tested have steel columns with grillage type connections to the slabs, and because of the lack of information to guide designers on this matter particular attention was given to the measurement of torsion in the legs of the grillage. The measurements were made by electrical or acoustic gages attached to the flanges of the grillage members, as it was assumed that any twisting of these members would be resisted by bending of the flanges, as discussed by Marsh.[3]

No significant torsion could be detected during these investigations and this result could probably have been anticipated from the study made by Wood[4] of composite action between slab and supporting beams. The extra reinforcement in column strips could be regarded as producing a supporting beam of flexibility close to that of half the slab width. The behavior of the slab could then be expected to approach the "twistless" case of the Grashof-Rankine analysis, in which there is no torsion in the slab.

An approach to the stress distribution in the neighborhood of an edge column, and which gives fair quantitative agreement with the results in slabs whose spans do not differ greatly is as follows.

A curve is drawn through the points of inflection of the two orthogonal column strips, the points of inflection having been determined by methods of rigid frame analysis. For square panels this curve will be approximately a semi-circle as shown in Fig. A.

*ACI JOURNAL, V. 32, No. 3, Sept. 1960 (*Proceedings* V. 57), p. 299. Disc. 57-14 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 9, Mar. 1961 (*Proceedings* V. 57).

†Division of Building Research, Commonwealth Scientific and Industrial Research Organization, Melbourne, Australia.

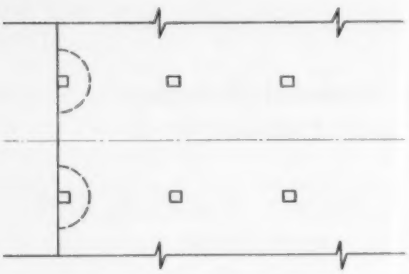

Fig. A—Stress distribution in neighborhood of edge column

It is then assumed that around the perimeter of this curve there is a uniform shear equal to the column load. There is no torsion present within this semicircular zone, if the moments due only to the flexural restraint imposed by the column are calculated by analogy with the stress distribution in an infinite half plate with a concentrated load on the boundary.[5] (See Fig. B).

This treatment ignores the influence of the load within the semicircular zone, but the measurements indicate that torsion from this source must be small or nonexistent.

It is apparent that the information reported here is not of itself a practical method of design, since it is intended to be a progress report. However, it is hoped that it will be used as a guide in the development of design methods.

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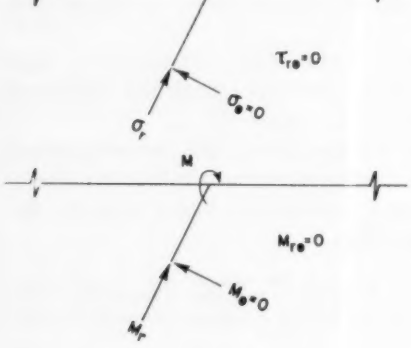

Fig. B—Stresses and moments on an infinite half plate

3. Marsh, C., "Rapid Design of Beams in Torsion," *Proceedings*, ASCE, V. 85, No. EM1, Jan. 1959, pp. 79-94.

4. Wood, R. H., "Studies in Composite Construction, Part II," *National Building Studies Research Paper* No. 22, Her Majesty's Stationery Office, London, 1955.

5. Timoshenko, S., *Theory of Elasticity*, McGraw-Hill Book Co., Inc., New York, 1934, p. 82.

By ROBERT R. KREPS and RAYMOND C. REESE*

The paper discusses a problem which has bothered designers for some time. The approach outlined in this paper, that whatever moment is not transferred by flexure into the face of the column must be developed by torsional shear in the slab along the sides of the column, is certainly indisputable. Whether the assumption of a prismoid sloping out to an additional width of $t - 1\frac{1}{2}$ in. from all sides is equally valid, is not as certain.

Having had a related problem a few years ago and having made tests of a sufficient number of small specimens to serve our purpose, we thought the tabulation of the results obtained might shed some light on the numerical values applicable to this problem.

Specifically, we desired to determine, for a flat plate of monolithic reinforced concrete exposed to lateral (seismic or wind) loads, the effective width of column strip available to transmit column moments into the slab. Nothing like the full width of panel to lines half-way between columns on each side of the column under study could possibly be considered. Similarly, a strip of slab equal to the width of column would be too narrow. The question, then, was: How many slab thicknesses on each side of the column could be safely counted on?

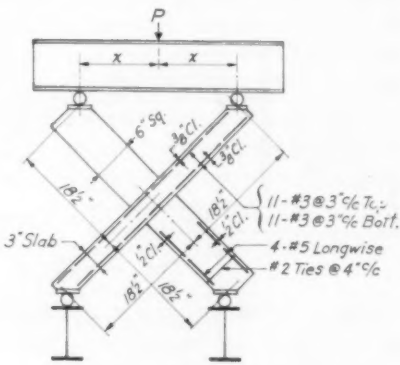

Fig. C—Inclined loading of Specimen 1

*Members American Concrete Institute, structural engineer and partner, respectively, Raymond C. Reese and Associates, Toledo, Ohio.

Six specimens were built, one as in Fig. C, five as in Fig. D. The size was determined by selecting small-sized, standard deformed bars and then proportioning the concrete around them. The size is large enough so that we are dealing with small specimens, not models, and free from problems of similitude.

Specimen 1, was loaded in an inclined position (Fig. C), in a simple rig, quite satisfactorily. Discussion suggested the loading procedure might involve small discrepancies, so Specimens 2 to 6 were loaded, in a vertical position (Fig. D). Strain measurements were made on the bars and deflections were taken from a wire grid stretched across the face of the specimen.

Table A gives salient information on the slab thickness, strength of control cylinders, number and arrangement of bars, load P, and applied moment Ph at failure. The next columns in Table A compute the width of concrete in the slab and the number of bars, to produce an ultimate resisting moment equal to that applied.

Since studying the theoretical analysis presented in the DiStasio-van Buren paper, we have computed the width of base of shear prismoid (b or c of the paper), using t — ½ in. because of the size of specimen. In the last column of Table A are values of v_t computed from Eq. 8. It will be noted that the values of v group around 450-550 psi, except for the low value for Specimen 5, which had the major part of its slab reinforcing grouped directly over the column, yet failed along a line a couple of slab thicknesses away from the column like the others. Thus it may be that more of the column moment was transmitted into the slab by torsional shear on the sides of the column than the computed value for v suggests and that less than the theoretical ultimate flexural capacity went out through the face of the column.

This is by no means a complete or thorough test program. It served its original function and is offered here for two reasons: (1) to see what

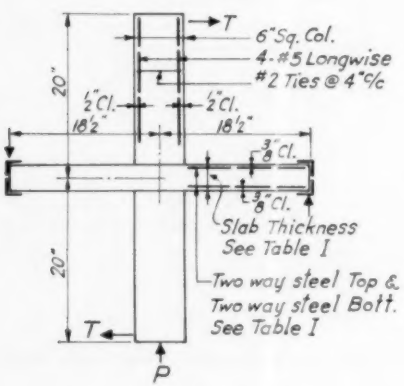

Fig. D—Vertical loading of Specimens 2 to 6

TABLE A

Specimen No.	Slab thickness, t, in.	f_c', psi	Reinforcing	Load P at failure, kips	Moment Ph at failure, in.-kips	Required effective width and number of bars for M_s	Base of shear prismoid C, in.	$v_t = \frac{Ph - M_1 - M_2}{J}\frac{c}{2}$ psi
1	3	3000	11-#3 @ 3 in. c/c	5.65	351	23.4 in. 8 bars	11	450
2	3	1750	11-#3 @ 3 in. c/c	7.28	352	30.6 in. 10 bars	11	500
3	2½	2470	11-#3 @ 3 in. c/c	4.78	212	24.4 in. 8 bars	10	460
4	4	2920	11-#3 @ 3 in. c/c	13.2	682	32.9 in. 11 bars	13	560
5	3	2450	11-#3 7 bars @ 1½ in. at column	6.73	321	23.5 in. 9 bars	11	152
6	3	2590	7-#4 @ 5½ in.	6.73	321	26.6 in. 5 bars	11	450

light it may shed on this problem when considered by other investigators, and (2) to illustrate the fairly simple procedure, which may attract others to carry the matter further.

AUTHOR'S CLOSURE

Interesting discussions have been presented by Messrs. Beresford and Blakey and by Messrs. Kreps and Reese. While the purposes of their investigations were somewhat different from that of the paper, they provide definite contributions to the general information on this subject.

Messrs. Beresford and Blakey deduce that if a section be taken through the points of inflection surrounding an exterior column and the shear be considered uniformly distributed thereon, the eccentric moment on the column will produce no torsion on the structure as a whole within this periphery. While it may be essentially true that there are no torsional stresses on radial and circumferential sections, there will of course be torsional stresses on all other sections not normal to the direction of the principal bending moments. The sides of the critical section parallel to the direction of the resultant applied moment will be predominantly in torsion and subject to shear variation as demonstrated in the paper. The determination of peripheral sections on which there is no torsion, although not parallel with the critical section, may provide a useful approach to the evaluation of stresses within the column head region.

The investigation by Messrs. Kreps and Reese was primarily intended to determine the effective width of slab available to resist a column moment. Their problem was therefore primarily one of bending rather than critical shear as in the paper. The present ACI Code specifies that

for an interior panel 76 percent of the moment is to be distributed to the column strip and 24 percent to the middle strip. Some designers prefer to take the entire moment in the column strip. As long as shear failure is not imminent, it is believed that either method should provide satisfactory results.

Some difficulty was experienced in reconciling the values of Table A. Apparently the load P represents a different quantity from P in Fig. 1A. The low value of 152 psi for the maximum torsional shear in Specimen 5 perhaps should read 352 psi which is about what might be expected from the formulas in comparison with the 450 psi of Specimen 6. Because of the bunching of the bars over the column in Specimen 5, more of the moment is carried directly into the column by bending, thus leaving a smaller value for torsion as affecting shear. This is in agreement with the theory presented in the paper. The fact that the specimens failed on a line several slab thicknesses away from the column may indicate that one-way beam action was a significant factor in the actual mode of failure. It is believed that an extended series of similar tests at a larger scale would verify the distribution of stresses in the column head region and form a basis for establishing safe values.

ACI NEWS LETTER

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ANNUAL REPORT – 1960

Toward More International Activities

International relations

Cooperation with organizations in other countries increased during 1960. The Institute was represented at a number of meetings, including the International Association for Bridge and Structural Engineering by Douglas McHenry, past president; Third International Congress of Precast Concrete Industry by Arsham Amirikian; and the RILEM Conference on Autoclave Cellular Concrete, Gothenburg, Sweden, by J. J. Shideler and John K. Selden. During the course of the year, 23 papers by overseas authors appeared in the JOURNAL.

CEB Collaboration Committee

The three-member Collaboration Committee with the Comité Européen du Béton, Eivind Hognestad, Raymond Reese, and Chester Siess, continued its active work in arranging for the translation, printing, and distribution of the published reports of the CEB. Most active organizations in providing these translations were the Cement and Concrete Association, London, England, and the Portland Cement Association. Two translations were prepared by ACI members and are in the process of being published. As a result of the meeting of the CEB in Monaco in January, 1961, the Collaboration Committee expects to review the method by which these translations are made available and perhaps recommend a system of condensation to provide the essential information without the expense and time necessary for complete trans-

lations. A translation of a paper by Dr. Hubert Rüsch, a member of the CEB, was published in the July 1960 JOURNAL. An additional paper by Prof. Franco Levi appears in the March 1961 JOURNAL.

Work of translating ACI standards and committee reports into other languages continues. The Committee on Spanish Translations consisting of Celso Carbonell, Panama; Adolfo Zeevaert, Mexico; and Edgardo Quintanilla, Peru, has reviewed and approved translations of the ACI Building Code, the report of ACI-ASCE Committee 323 on prestressed concrete, the *Reinforced Concrete Design Handbook*, and the ACI recommended practice for hot weather concreting, into the Spanish language.

Preliminary arrangements have been made for the translation of some ACI standards into Turkish. The *Concrete Primer* has been translated into Italian and published in that language.

Finances

Over-all financial results for 1960 were improved over 1959. Total income increased by about 13 percent as shown in Fig. 1. ACI members dues accounted for 55.4 percent of total income, compared to 52.4 percent in 1959 whereas income from publications declined to 35.9 percent from 38.5 percent in the previous year, and 42.6 percent in 1958. The increase in dues rate was largely responsible for the increase in total amount received from this source. Increase in dollar income from publication sales is due almost entirely to advance receipts for the ACI 55-Year Index.

Membership

Table 1 summarizes membership activities for the past 8 years and indicates a continuation of the trend of the past 5 years in that while total membership is growing the rate of growth is declining. Total applications in-

Fig. 1—Sources of ACI income, 1954-1960

TABLE 1—SUMMARY OF MEMBERSHIP GAINS AND LOSSES, 1953-1960

Year	Losses				Applications and reinstatements	Net Gain	Active at year end
	Deaths	Resignations	Non-payment of dues	Total			
1953	28	171	395	594	974	380	6352
1954	23	159	367	549	896	347	6696
1955	26	159	396	581	1269	688	7387
1956	32	153	389	584	1641	1057	8444
1957	28	193	428	649	1428	779	9223
1958	29	219	592	840	1340	500	9723
1959	43	258	595	896	1283	387	10,110
1960	47	313	728	1088	1326	238	10,348

creased but net gain continued to decrease with a small increase in number of members lost through nonpayment of dues and by resignation.

Publication sales

Publication sales changed little in 1960 from the previous year and would have ended up about equal to 1959 except for advance receipts on the 55-Year Index. A number of factors account for this static situation among them being the fact that ACI principal publications are 2 or more years beyond new editions and therefore are experiencing an expected gradual decline in sales. Table 2 and Fig. 2 give sales for the five principal ACI publications plus the 55-Year Index.

Publications

Recognizing the importance of the newly authorized monograph series, the Technical Activities Committee set up a suborganization, the TAC Committee on Monographs; G. E. Burnett,

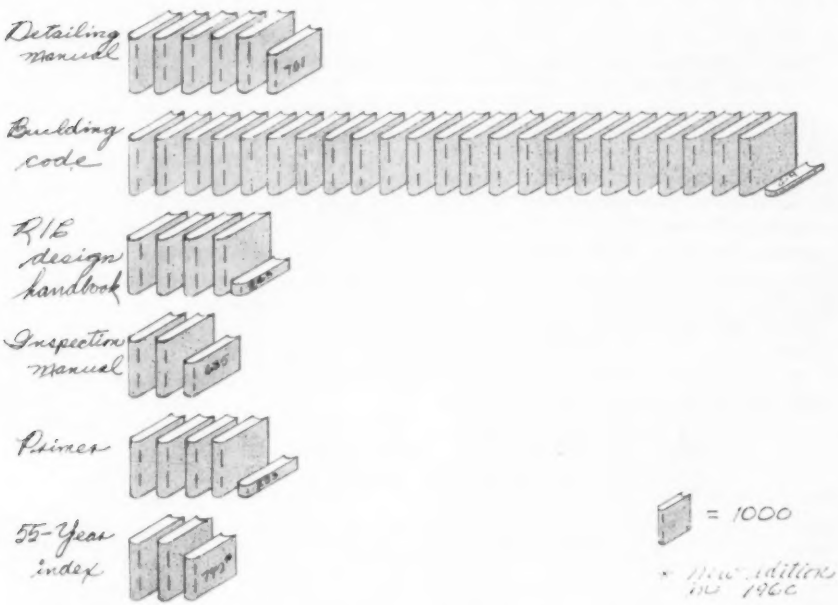


Fig. 2—Sales of ACI special publications, 1960

TABLE 2—UNIT SALES OF ACI SPECIAL PUBLICATIONS, 1955-59

	1955	1956	1957	1958	1959
Detailing Manual	6,031	7,430	11,472*	8,385	6,100
R/C Design Handbook	5,108*	5,265	4,181	3,765	4,165
Building Code	16,619	60,653*	20,659	27,775	26,725
Inspection Manual	6,619*	4,266	2,926*	4,318	4,167
Primer	797	1,032	1,840	4,599*	3,776

*New editions in these years

Chairman; to supervise this operation. The mission established by TAC is:

> "The mission of the TAC Committee on Monographs is to expedite the dissemination of knowledge of specialized facets of the concrete field through the development of a series of monographs for publication by the Institute.
>
> "In furtherance of this mission, the TAC Committee on Monographs will consider the available literature and determine the need for brief authoritative treatises on specialized segments of knowledge in the concrete field, select and negotiate with authorities in these areas of knowledge for writing of the text, and moderate the critical review of an approved finished work for publication."

The TAC Committee on Monographs met in Tucson at the time of the 1960 regional meeting and laid the ground work for an orderly procedure for the selection of subjects and author, approval, negotiation with authors, payment schedules, final commitments, and editorial procedures

At the end of the year, definite commitments were made with 15 authors on a variety of subjects. One manuscript was in process of being distributed to members on monograph committee for review and a second was virtually completed with the first draft having been reviewed and the author preparing the final draft. Other monograph manuscripts are expected during 1961.

55-Year Index

Editorial work on the ACI 55-Year Index was completed under the supervision of Robert G. Wiedyke, then associate editor. Final proofs for all of the text were approved during the year but completion of the printing and binding and distribution of the book was delayed until early 1961. Preliminary orders offered to members at the introductory price of $3.50 amounted to slightly over 3000 at the time this report was written, in early January, 1961. The regular selling price, after introduction, will be $4.50 to members and $9.00 to nonmembers of the Institute. This publication will be a major aid to users of ACI literature and will supplant a number of partial indexes which had been formerly used and also will cover 25 years of *Proceedings* which were never included in the cumulative indexes.

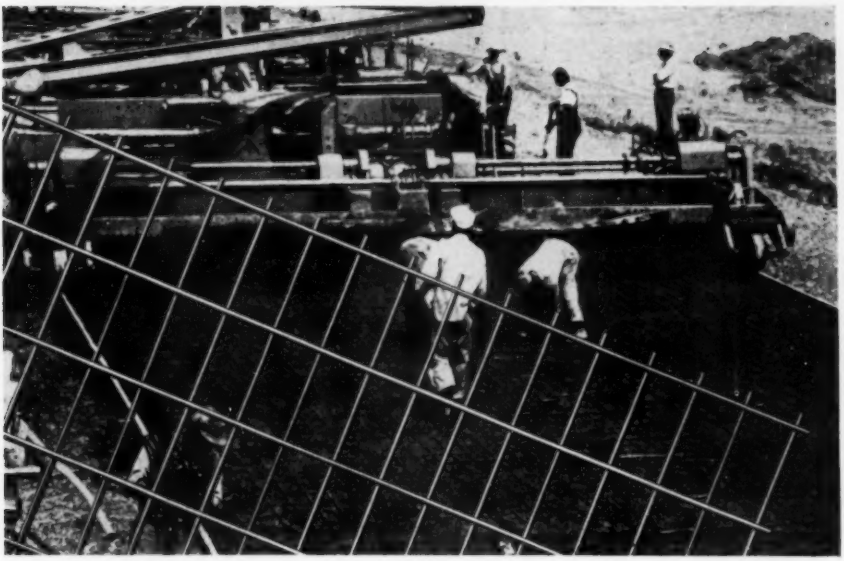
Mrs. Mary K. Hurd, former associate editor of the ACI JOURNAL, is working on a part-time basis in cooperation

Continued on page 24

TABLE 3—ADVERTISING SALES AND EXPENDITURES

Year	Pages, No.	Gross Sales	Expenses	Net
1956	146½	$25,011.54	$11,504.42	$13,507.12
1957	149⅓	25,013.18	14,426.24	10,586.94
1958	151¼	33,440.07	16,537.60	16,902.47
1959	144⅗	26,431.01*	11,072.24*	15,360.77
1960	151½	26,837.29*	10,960.46*	15,876.83
1961 (Est)	122	25,000.00*	10,500.00	14,500.00

*Does not include agency commissions



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Concreting Esquatzel Canal*

■ TEN MILES OF STANDARD railroad-size rail were laid in preparation to concreting the Esquatzel Diversion Canal in southeastern Washington. There were three rails; the middle one was put at the bottom of an 18½ ft deep ditch.

The Esquatzel Diversion Canal — a winding concrete furrow which cuts across the barren plain of the Columbia River Basin is designed to release drainage water from the lower part of the Columbia Irrigation District, as well as to prevent flash floods in the Pasco area.

The rail was a key element in the automated concreting of the gaping canal. On it moved the ponderous trimmer, slip form, and screeding machines (See Fig. 1) which enabled the contractor to concrete 900 to 1000 ft of one side of the canal per 8-hr shift.

Constructed at a cost of $2,700,000 for the U. S. Bureau of Reclamation (Columbia Basin District), the canal measures 64 ft across at the top, 8 ft across at the bottom. Slope of the sides is 1½:1, or approximately 34 deg.

Excavation of 1,300,000 cu yd of earth preceded placing of the 4½-in., nonreinforced concrete lining.

The contractor had up to 120 men on the job, which lasted 1 year, and was finished in December, 1958.

About 66,000 cu yd of concrete went into the Esquatzel Canal; of that total some 6000 cu yd were ready-mixed the rest dry batched.

The general contractor for the project was Donald M. Drake Co., Portland, Ore. Concrete was supplied by Pre-Mix Concrete, Inc., Kennewick, Wash.

*Abstracted from *This Earth*, V. 12, No. 2, Feb. 1959.

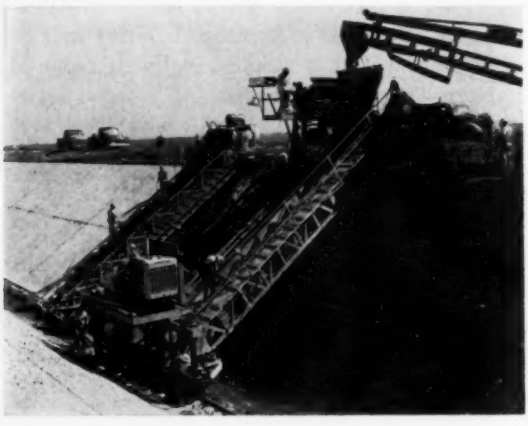

Fig. 1—Automated concreting of the Esquatzel Diversion Canal near Pasco, Wash. The slip form and screeding machines approaching from the left, leaving behind a swath of finished concrete. At top, "dry batch" is being fed the machine by mobile conveyor. A trimmer machine trimmed the side of canal in advance of the slip form

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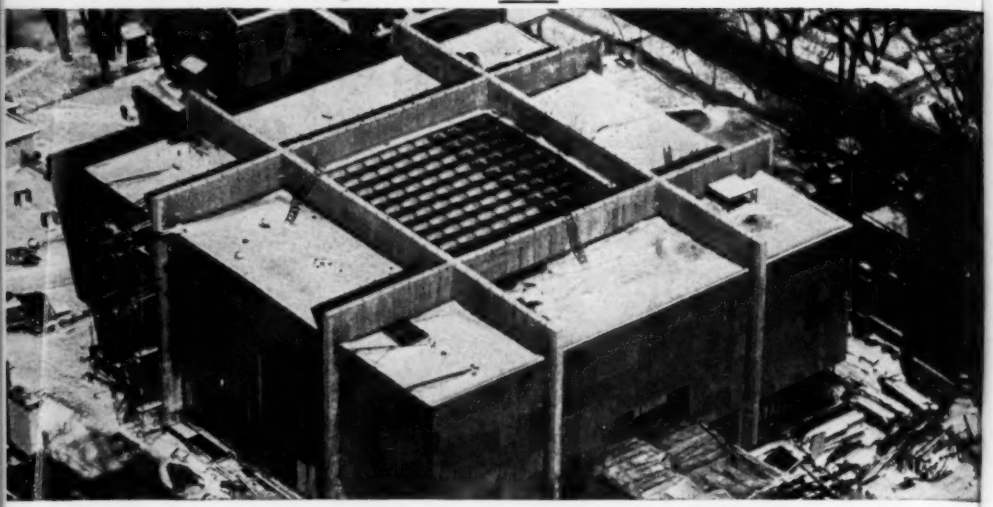

Unique structural system for Utica's newest art gallery

The huge monolithic, intersecting prestressed girder system shown here is the heart of a unique engineering and construction achievement. In a continuous 16-hour placing operation, the girder was cast in place fifty feet above grade. Dr. Lev Zetlin's mix design included POZZOLITH to provide concrete meeting these rigid requirements:

- 28-day strength of 5000 psi.
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- Extremely high degree of workability to prevent voids and honeycombing in girders crowded with reinforcing steel and post-tensioning cable conduits.
- Lowest possible unit water content.

In addition, POZZOLITH was used in all job-placed concrete—for foundations, walls and exterior supporting columns to obtain increased workability and minimum shrinkage.

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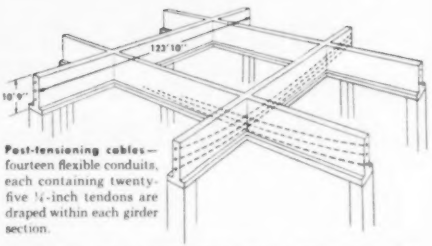

Post-tensioning cables—fourteen flexible conduits, each containing twenty-five ¼-inch tendons are draped within each girder section.

DESIGNED BY ARCHITECT PHILLIP JOHNSON AND ENGINEERED BY DR. LEV ZETLIN—this concrete tic-tac-toe provides an interior free of columns for Munson-Williams-Proctor Institute Art Gallery in Utica, New York. Contractor: George A. Fuller Company, New York; Supervising Architects, Bice and Baird of Utica. POZZOLITH ready-mix concrete from American Hard Wall Plaster Company, Utica.

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Positions and Projects

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 207, Properties of Mass Concrete

O. C. Zienkiewicz
Northwestern University
Evanston, Ill.

Committee 216, Fireproofing or Fire Protection of Structures

Charles F. Peck, Jr.
Ceco Steel Products Corp.
Chicago, Ill.

Committee 325, Structural Design of Concrete Pavements for Highways and Airports

Phillip L. Melville
Office, Chief of Engineers
Department of the Army
Washington 25, D.C.

Committee 403, Adhesives for Concrete

Freeman P. Drew
Association of American Railroads
Chicago, Ill.
Belmon U. Duvall
Corps of Engineers
U. S. Army
Cincinnati, Ohio
Elmo C. Higginson
U. S. Bureau of Reclamation
Denver, Colo.
James D. Kreigh
University of Arizona
Tucson, Ariz.
Herbert A. Rooney
California Division of Highways
Sacramento, Calif.

Committee 622, Formwork for Concrete

Paul S. Barton
Form Designer
Birmingham, Mich.

Committee 622 reports on "Formwork for Concrete"

Culminating more than 6 years of work, Committee 622 presents the committee report on "Formwork for Concrete" found on p. 993 of this month's JOURNAL.

The committee was originally organized in 1955, comprised of men selected for their knowledge and experience in planning and constructing formwork and concrete structures. The committee was assigned the task of working up a recommended practice for concrete formwork in general terms, based on considerations of safety and appearance. Ten subcommittees were formed to study and report on specific phases of the problem.

Subcommittee II's report, "Form Construction Practices," was published in the June, 1957, JOURNAL. A preliminary report, "Pressures on Formwork," was published in the August, 1958, JOURNAL.

Numerous subcommittee reports, contributions of committee members, an analysis of replies to a nationwide questionnaire, and a thorough study of the pertinent material listed in a bibliography compiled by the committee, were all used in compiling the present report, which, following discussion, will form the basis of a recommend practice, the committee's assigned task.

Harry Ellsberg, consultant, Giffels and Rossetti, Detroit, has served as chairman of the committee since its inception. Committee members working with Mr. Ellsberg in compiling this report were: John Banker, construction consultant, Portland Cement Association, Chicago; Frank H. Beinhauer, J. L. Simmons Co., Inc., Decatur, Ill.; H. P. Cerutti, manager and chief engineer, Steel Forms Department, Blaw-Knox Co., Pittsburgh; N. L. Doe, vice-president, Turner Construction Co.,

ACI Southern California Chapter and the Southern California District Council of American Society for Testing Materials held a joint meeting on January 19. A. Allan Bates, vice-president of research and development, Portland Cement Association, Chicago, who holds a high place in the official ranks of both ACI and ASTM, was featured speaker for the meeting. Shown in the photo, reading from left to right are: ACI Southern California Chapter President Dewain R. Butler, president, Integrated, Inc., Los Angeles; Chapter Vice-President Byron P. Weintz, chief engineer, Consolidated Rock Products Co., Los Angeles; Dr. Bates, ACI director and ASTM president; T. A. Marshall, Jr., executive secretary ASTM; and John B. Howe, vice-chairman, Southern California District Council ASTM.

New York; Jacob Feld, consulting engineer, New York; David E. Fleming, vice-president and chief design engineer, A. J. Ryan and Associates, Inc., Denver; and Vance J. Gray, structural engineer, Raymond C. Reese, Toledo.

Other working committee members are: Robert C. Johnson, president, Siesel Construction Co., Milwaukee; Victor F. Leabu, assistant chief structural engineer, Giffels and Rossetti, Detroit; Verne O. McClurg, partner, McClurg, Shoemaker and McClurg, Chicago; Donald R. Peirce, Ammann and Whitney, New York; A. H. Pilling, Richmond Screw Anchor Co., Inc., Brooklyn; Joseph R. Proctor, Jr., Dravo Corp., Pittsburgh; Paul F. Rice, technical director, Concrete Reinforcing Steel Institute, Chicago; O. G. Sharrar, general superintendent, O. W. Burke Co., Detroit; N. L. Stephens, president and general manager, Dixie Form and Steel Co., San Antonio; P. R. Stratton, Hydro-Electric Power Commission of Ontario, Toronto, Ont., Canada; William R. Waugh, chief, concrete branch, Engineering Division, Civil Works, Department of the Army, Washington, D. C.; and W. H. Wolf, U. S. Bureau of Reclamation, Denver.

In addition to the recommended practice, the committee has on its agenda the writing of a manual on concrete formwork, the initial work for which is already underway.

ACI members open Puerto Rico office

Ignacio Martin announces that he has moved to San Juan, Puerto Rico to open an engineering office under the firm name of Sacmag of Puerto Rico, Inc. He will be associated with J. L. Capacete and V. Ramos in the Puerto Rico office.

Luis P. Saenz continues in their Havana office in the engineer-architect firm, Saenz-Cancio-Martin-Gutierrez.

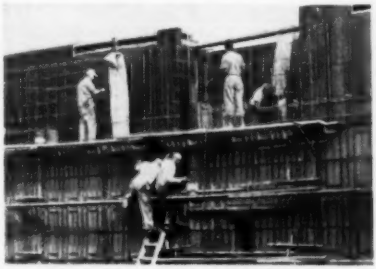
ACI 55-Year Index off the press

The latest ACI publication, the "55-Year Index, 1905-1959," a guide to more than a half-century of important writings in concrete progress, rolled off the press last month with close to 3000 advance orders ready to be filled. This comprehensive book provides between two covers an index of all the material published in the ACI *Proceedings* from 1905-1959. It includes previously indexed material from the ACI JOURNAL, *Proceedings* V. 26-55, plus material heretofore not indexed from *Proceedings* V. 1-25, which appeared solely as single-volume convention reports.

The new index contains about 18,000 entries. The major headings, the number of which has been reduced from previous indexes with a liberal use of cross references, are in bold type for aid in locating material quickly. Each paper is indexed under the title, author, and major subject headings. Published discussions are noted with individual discussers listed individually. The informative brief discussions which have appeared as "Job Problems and Practices," "Letters from Readers," "Concrete Briefs," and "Problems and Practices" are indexed under their main subjects. Published miscellany of special interest to ACI members such as committee reports and presidential adresses are also included. A special synopses section is devoted to brief summaries of all papers published in the JOURNAL since its inception in 1929.

The assembling of a comprehensive index of the ACI *Proceedings* began in 1947 with the 10-Year Index of JOURNAL papers, 1937-1947. To this was added the index of JOURNAL papers of 1947-1949 and 1929-1938. The first index required only assembling separate indexes but the 20-Year Index required detailed indexing of the earlier material. This was also true for the material destined for the over-all compilation, that published prior to 1929—the 25 years before the JOURNAL came into being. The slow process began soon after the completion of the 20-Year Index but due to limited staff and funds had to be delayed. A 5-Year supplemental index was assembled from individual indexes for 1950-1955 when the delay had to be extended. The laborious work got under way again about 2 years ago. In addition to the new indexing, the task required a great deal of time to assemble from the various sources previously indexed material and arrange the book in consistent, usable style with a minimum of duplication.

The new index, in conjunction with the V. 56 index published in the Part



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FREEZE-THAW TESTING NOW AUTOMATED

TESTING THE EFFECTIVENESS of entrained air in combating freeze-thaw damage to concrete has come a long, long way in the past few decades. By all odds, however, the most important step forward ever taken in this phase of concrete testing is a new "weathermaking" machine for laboratory use, the Conrad Freeze-Thaw Chamber. To help you appreciate its importance, let's go back and take a brief look at some of the stages of development in freeze-thaw testing.

Back in the days when no ASTM test methods for evaluating freeze-thaw resistance were available, we resorted to a pretty basic technique. Solid concrete blocks were cast in a special mold to provide a dam which would hold calcium chloride solution on the surface of the concrete. These blocks were then set on racks in an area outside the lab to let nature take its course. New England weather enabled us to record an average of 40 cycles of freezing and thawing in a season. This was a slow process and in many instances, it took more than one season to come up with any significant results.

The next step was to develop a laboratory method, using a cold-storage chamber for freezing, which gave us some control of the cycles. This method was essentially the same as the present ASTM Designation C 292. During the first few years, it was necessary to rely on visual examination of the specimens, and/or weight loss for evaluation. However, this technique served its intended purpose in many thousands of tests.

Late in the 1930's, the use of sonic vibration was suggested. We constructed several sonic modulus units for use in other laboratories, as well as in our own, and this method of comparing specimens proved to be a major advance in the development of nondestructive testing of concrete.

Now, however, the new and completely automatic Conrad Freeze-Thaw Chamber recently added to our laboratory facilities opens a whole new era of testing speed and efficiency. One of the few privately owned machines of its kind anywhere in the world, this apparatus will produce as many as eight complete freeze-thaw cycles in a single 24-hour day. Each cycle will bring the interior temperatures of the test specimens from 40F. to 0F. to 40F. Thermocouples imbedded in the specimens deliver interior temperature readings. Test series can be completed in twenty to thirty days instead of the ten to fifteen months required by earlier methods.

New Conrad Freeze-Thaw Chamber in operation at Dewey and Almy concrete testing laboratory.

The unit is capable of automatically duplicating cycles in compliance with all four ASTM designations, C 290, C 291, C 292 and C 310. Controls can be adjusted to obtain practically any combination of time-temperature cycles. This flexibility permits investigation of many types of cycles which may, in some ways, more closely approach those actually encountered in the field.

Thus, this new "weathermaking" machinery will serve as an important research tool, as well as a superior testing facility for investigating concrete performance under standard ASTM conditions.

2, December, 1960, JOURNAL, brings to the industry a complete index of the ACI *Proceedings*. With the indexing of each volume as it is published, as is now the practice, readers of the ACI JOURNAL will always have at their fingertips a quick way of "retrieving" the pertinent information they require.

The 55-Year Index, of 6 x 9 format, contains 364 pages in hard covers with standard library binding that will lie flat on the desk when opened. The cover is of vinyl impregnated cloth which is both durable and attractive. The Index is available at $9.00 ($4.50 to ACI members) from Institute headquarters.

Ferguson elected TSPE president

ACI past president Phil M. Ferguson, professor of civil engineering, University of Texas, Austin, was recently elected president of the Texas Society of Professional Engineers. A charter member of TSPE and Travis Chapter, Professor Ferguson has held a number of administrative posts in both groups. Last year he served as second vice-president of TSPE.

After graduating from the University of Texas in 1922 and receiving MS degrees from the University of Texas in 1923 and the University of Wisconsin in 1924, he spent several years with Dwight P. Robinson and Co., New York, in the structural design of power plants and industrial buildings and the construction of hotels and apartment buildings.

Professor Ferguson joined the University of Texas faculty as associate professor in the field of structures and materials in 1928. He became professor in 1939 and was made chairman of the civil engineering department in 1943, a post from which he retired in 1957. Long recognized as an authority on structural design, Professor Ferguson has been consultant on a number of projects. He has authored many research papers and a recent textbook, *Reinforced Concrete Fundamentals*, has become a technical bestseller.

Active in ACI since 1930, Professor Ferguson is currently a member of ACI Committee 115, Research; Committee 208, Bond Stress; and Committee 318, Standard Building Code. He is ACI representative on the American Welding Society Committee on Welding Concrete Reinforcing Steel.

Corbetta and Lewis on NCCA board of directors

The National Concrete Contractors Association recently held its 4th annual convention at the Diplomat Hotel, Hollywood-by-the-Sea, Fla. Names of two prominent ACI members appear on the NCCA board of direction for 1961: Roger H. Corbetta, president, Corbetta Construction Co., Inc., New York, and Elbert F. Lewis, president, F. D. Lewis & Son, Inc., Greensboro, N. C.

Contractors from all over the country were interested in the program which featured a talk on "Washington Trends" presented by Vincent Ahearn, National Sand and Gravel Association and National Ready-Mixed Concrete Association.

Charles E. Proudley, director of engineering and executive secretary of Carolinas Ready-Mixed Concrete Association, spoke on "Concrete Curing." Robert A. DeRose, vice-president and general manager of Paddock Pools, Van Nuys, Calif., was also a featured speaker. Mr. DeRose is credited with having designed a mechanical installation for the world famous pool at the University of Mexico. He spoke on "Building Swimming Pools for Profit."

Williams appointed ACPA representative

Jack C. Williams has been appointed Western Region representative for the American Concrete Pipe Association, Chicago, and will headquarter at the ACPA Western Region office in Palo Alto, Calif.

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Clair awards ACI memberships to promising engineering students in three universities

For the past 7 years Miles N. Clair, president, Thompson-Lichtner Co., Inc., Brookline, Mass., has been the anonymous donor of funds for ACI student memberships. Establishing funds for this purpose in the name of Northeastern University, Boston; Tufts University, Medford, Mass.; and Drexel Institute of Technology, Philadelphia, he has sponsored and contributed memberships for 20 promising engineering students, recommended by their respective schools.

Mr. Clair has given unstintingly of his time and energy for many years and has been awarded numerous citations for his civic leadership, his liason work with senior engineering students, and for advancing the technology of concrete construction.

An ACI member since 1925, Mr. Clair has authored numerous JOURNAL papers and has also been active on both administrative and technical committees of the Institute. He is currently ACI representative on the ASA Sectional Committee (A1), Specifications and Methods of Test for Hydraulic Cements.

NSGA and NRMCA hold annual conventions

The 45th annual convention of the National Sand and Gravel Association and the 31st annual convention of the National Ready Mixed Concrete Association, Washington, D. C., were held Jan. 23-26, 1961 at the Hotel Americana, Bal Harbour, Miami Beach, Fla.

Technical sessions opened Monday, January 23, with about 25 papers presented at five sessions. Presentations included a "Review of Sand and Gravel Research Problems" and a report on "Investigation of Methods for Measuring Mixing Efficiency of Truck Mixers." Other technical papers covered winter concreting problems, admixtures for concrete, control of ready-mixed concrete and a progress report on "Variations in Portland Cement."

At a clinic on technical problems questions from three categories: (a) Standard testing procedures for aggregates and concrete; (b) Calculating proportions for concrete; (c) Special problems of specifications, uses, and acceptance, furnished material for round table discussions.

PCA offers fellowships for graduate students

The Portland Cement Association has made 20 fellowships to the Graduate School of Purdue University available for the summer session starting June 19 and ending August 11. The free-grant awards will benefit qualified instructors and members of civil engineering departments with special interest in highway planning, design, and portland cement concrete technology. In addition to payment of tuition, fees and transportation expenses, fellowships also include a cash stipend of $700.

The study program, which will include courses in concrete and aggregates, highway planning, and economics and pavement design, will be supplemented by a symposium conducted by outstanding authorities in highway engineering.

Application forms and other information concerning the fellowships can be obtained from K. B. Woods, professor, School of Engineering Purdue University, Lafayette, Ind.

Haas to lecture at University of Texas

A. M. Haas, professor of concrete and concrete construction, Institute of Technology, Delft, Netherlands, will serve as visiting professor in a graduate course on thin shell design to be presented in the spring semester at the University of Texas, Austin.

Dr. Haas is widely known and internationally active. He is a member of the European Committee on Concrete

(Comité Européen Béton) which is attempting to stimulate and correlate research and thus create a common base for the building codes in the different countries. He is a member of the executive committee of the International Association for Shell Structures. He has been president of the Netherlands Concrete Association and the Netherlands Committee for Concrete Research. He is a member of ACI and the International Association for Bridge and Structural Engineering.

Dr. Haas will lecture daily on thin shell design as the middle portion of the graduate course, Advanced Reinforced Concrete Design. This course under ACI past presidest Phil M. Ferguson, professor, Department of Civil Engineering, University of Texas, will be devoted to the design of folded plates, barrel shells, and double curvature shells, including the hyperbolic paraboloid.

Ferguson to receive ASCE research award

ACI past president Phil M. Ferguson, professor of civil engineering, University of Texas, Austin, is one of five prominent engineering educators, to be nominated for the ASCE research prizes for 1960. Awards will be made at the Annual Research Luncheon to be held on April 12, during the ASCE Phoenix Convention.

The winners in addition to Professor Ferguson are Raymond Clough, professor of civil engineering, University of California, Berkeley, Calif.; Donald R. Harleman, associate professor of hydraulics, Hydrodynamics Laboratory, Massachusetts Institute of Laboratory, Massachusetts Institute of Technology, Cambridge; Bruno Thürlimann, professor of civil engineering, Swiss Federal Institute of Technology, Zurich, Switzerland; and David K. Todd, associate professor of civil engineering, University of California, Berkeley.

The ASCE Research Prizes were established in 1946 to stimulate research in civil engineering. Up to five awards are made annually to members of the society for notable achievements in research related to civil engineering. Each award consists of $100 and a suitable certificate.

Ahearn rejoins NSGA staff

Vincent P. Ahearn, Jr., has rejoined the National Sand & Gravel Association, Washington, D.C., as assistant to the associate managing director. Mr. Ahearn served on the NSGA staff for some time prior to his entry into the service where he has been for the past 3 years.

Cerro announces firm name change

The name of Cerro de Pasco Sales Corp., a subsidiary of Cerro de Pasco Corp., New York, has been changed to Cerro Sales Corp.

Rutherford and Chekene form engineering firm

John Brisbin Rutherford, structural engineer of Los Altos, and Constantine C. Chekene, structural engineer of San Francisco, announce the formation of a partnership known as Rutherford and Chekene, consulting engineers, with offices in San Francisco and Los Altos. The firm will specialize in site engineering and the design of structure, fields in which both partners have had long experience.

Mr. Rutherford, an engineering graduate of Lehigh University and California Institute of Technology, has worked for several San Francisco consultants and was head of the structural engineering department at Western-Knapp Engineering Co., before setting up his own practice in Los Altos which he has maintained for the past 3 years.

Mr. Chekene is a graduate of Harbin University, China. He too has worked as a structural engineer for several San Francisco engineering consultants, as well as for John Lyon Reid, architect.

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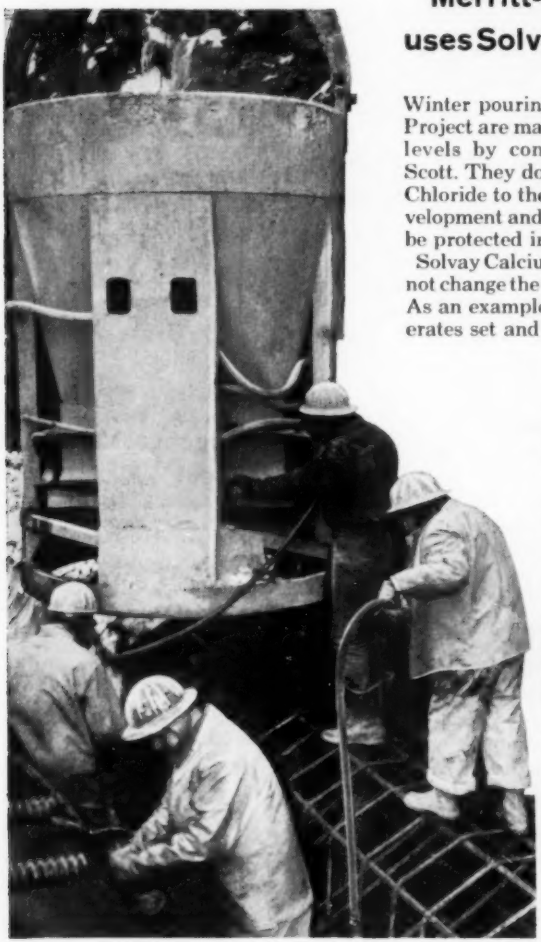

Winter pouring schedules at Niagara Power Project are maintained at near warm weather levels by contractor Merritt-Chapman & Scott. They do it by adding Solvay® Calcium Chloride to the mix to accelerate strength development and shorten the time concrete must be protected in freezing weather.

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Rogers opens Paris office

Paul Rogers & Associates, Inc., consulting engineers, Chicago, have announced the opening of their European office, Eugene Bonnet & Paul Rogers, consulting engineers, Paris, France.

The firm will offer full coordinating and consulting service to American industries abroad, specializing in the structural design of high rise apartment and office buildings, special structural systems such as prestressed steel and concrete, lift slab construction, and unusual foundations.

The firm will offer complete design of industrial plants, bridges and sanitary engineering structures, airplane hangars, grain elevators, parking garages, and television and radar towers.

Prestressed concrete to soar in 1961

Sales of prestressed concrete in the next 12 months are expected to increase approximately 20 percent over the present $300 million annual volume, based on expanded government road and bridge building projects, increased private plant and home construction and more varied uses for the product uncovered recently through intensive research.

Prestressed concrete first found acceptance in the United States because of a shortage of raw materials, particularly structural steel, during the Korean situation. However, since then the materials and techniques have stood on their own, and are widely used in many types of buildings and structures. In some cases, prestressed concrete has been 15 percent less in cost.

Jacob O. Whitlock, president of the Prestressed Concrete Institute and president of the Midwest Prestressed Concrete Co., Springfield, Ill., points out that major building codes and building supervisors of major cities have approved the use of prestressed concrete and that it has been widely demonstrated that it definitely is possible and practical to employ modern methods and materials entirely within the framework of such codes.

Looking ahead, projecting on the advances made during the past 10 years which will accelerate as new concepts, new methods, and new techniques are developed, there is a clear indication that gross annual sales can and will increase 20 percent per year through 1965 to an industry volume of nearly $1¼ billion according to Mr. Whitlock.

Dempsey joins Novak firm

John G. Dempsey, formerly president of Prestcrete Corp., Plano, Ill., is now director of engineering for Robert L. Novak and Co., Des Plaines, Ill., a consulting and testing agency specializing in the problems of concrete and asphalt.

J-M Research realigns Celite department

A realignment of the Celite Research Department at the Johns-Manville Research Center in Manville has been announced by G. B. Brown, director, Division of Research-Development.

Carl R. Vander Linden, who has been chief of the Celite product development section since 1953, has been named manager of Celite Research. Dr. Vander Linden succeeds H. L. King, Jr., who was recently appointed manager of the New Business Development Department at the Center.

Herman C. Fischer has been appointed chief of the Cement Additives Section. Dr. Fischer, who joined the J-M Research staff in 1957, was granted a doctorate in building materials and colloid chemistry at MIT in 1954.

Other appointments named Francis B. Hutton to head up the Celite Filtration Section; Raymond G. Riede, chief of the Celite Fillers Research Section at the Center; and Laurence E. Weymouth, research technologist.

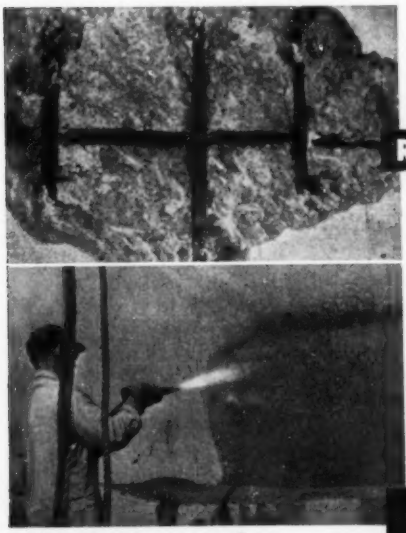
Conference on fundamental research in plain concrete

Arrangements are being made for the Second Conference on Fundamental Research in Plain Concrete to be held at the University of Illinois Conference Center, Allerton Park, near Monticello, Ill., from Sept. 5 to 8, 1961.

The success of the First Conference, held in September, 1958, has attracted much attention from persons prominent in the field of concrete research throughout the United States and abroad. Many requests for a second conference have been received.

The conference will afford an opportunity for discussion at the fundamental level of such topics as Kinetics of Hydration, Rheology of Concrete, Origin and Nature of Strength, Fracture, and Aggregate Reactions. The conference is intended to be an important means of disseminating information and ideas to an extent that cannot otherwise be achieved, either through the use of channels of publication or the presentation of papers at scientific meetings.

The organizations sponsoring the conference and their representatives are as follows: American Concrete Institute, Bryant Mather, supervisory civil engineer and chief of the Special Investigations Branch, Concrete Division, U. S. Army Engineer Waterways Experiment Stations, Corps of Engineers, Jackson, Miss.; American Society of Civil Engineers, Leo H. Corning, chief consulting structural engineer, Portland Cement Association, Chicago; American Society for Testing Materials, Kenneth B. Woods, head, School of Civil Engineering, Purdue University, W. Lafayette, Ind.; Portland Cement Association, Hubert Woods, director of Research, PCA, Chicago; Reinforced Concrete Research Council, Gene M. Nordby, head, Civil Engineering Department, University of Arizona, Tuc-



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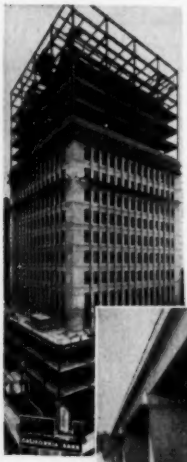

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son; and the University of Illinois, Clyde E. Kesler, professor, Department of T and AM, University of Illinois, Urbana.

The conference is supported by the National Science Foundation. The representative of each sponsoring agency serves on the conference Steering Committee with Professor Kesler as chairman. John W. Murdock, professor, Department of T and AM, University of Illinois, Urbana, is secretary to the Steering Committee. Complete details are available from Professor Kesler, conference chairman.

Inadequate financing stymies overseas construction

The National Constructors Association at its annual meeting held in New York, in January, urged the government to provide credit guarantees that would enable American construction firms to compete effectively in overseas markets.

Since World War II, the United States has actively encouraged private investment directed toward the reconstruction of developed countries and economic development of underdeveloped nations. While much progress has been made there is a continuing need for the kind of technical and managerial knowledge that American engineering and construction firms can provide, especially in Asia, Africa, and Latin America.

Another urgent need is a single source of information about construction opportunities overseas; and increased assistance from foreign service officers in United States Embassies and Legations.

National Constructors Association, with headquarters in Washington, is an organization of major engineering and building firms engaged in the design of chemical plants, petroleum refineries, power plants, steel mills and similar industrial facilities. All of the members operate on a nation-wide basis, and many work around the world.

BC Cement awarded PCA Safety trophy

The British Columbia Cement Co. plant at Bamberton, Vancouver Island, recently received its fifth safety trophy from the Portland Cement Association for a completely accident free year in 1960.

The safety record of the company is a result of personal attention of top management according to Arthur Lowe, personnel and safety supervisor at Bamberton. Suggestions for eliminating hazards are welcomed from the employees and every accident recorded by every cement plant in North America is studied as an object lesson. Each department holds monthly meetings on safety with the foreman of the department as chairman.

Visible result of this unremitting attention to safety detail is the concrete trophy standing on the lawn of the administrative building. Presented by PCA, it shows in bas-relief a figure with the motto "Safety Follows Wisdom" and chiselled beneath it is the record of five separate awards, each earned for an accident-free year.

Industry-wide construction equipment exposition planned

Preliminary arrangements have been completed for the first industry-wide Construction Equipment Exposition and Road Show to be held in Chicago in 1963.

The International Amphitheatre will be the site of the exposition, which sponsoring groups say will be the world's largest indoor industrial exhibition.

Previously known as the ARBA Road Show and limited to sponsorship of the American Road Builders' Association, the 1963 exposition will include sponsors from other influential segments of the road building construction equipment industry.

It will be produced and managed by the Construction Industry Manufacturers Association. In addition to ARBA and CIMA, sponsors will include Associated General Contractors of America, International Road Federation, and Associated Equipment Distributors.

Annual meetings of several of the sponsoring and allied associations are being planned to concur with the Chicago exposition.

LOOKING AHEAD

Mar. 5-8, 1961—Annual Convention, American Road Builders' Association, Chalfonte-Haddon Hall, Atlantic City, N. J.

Mar. 13-16, 1961 — 53rd Annual Convention, American Concrete Pipe Association, Hotel Americana, Bal Harbour, Fla.

Mar. 24-25, 1961—Annual Convention, Midwest Ready Mixed Concrete Association, Drake Hotel, Chicago

Apr. 10-14, 1961—Phoenix Convention, American Society of Civil Engineers, Hotel Westward Ho, Phoenix, Ariz.

May 11-13, 1961—Annual Convention, National Lime Association, Grand Hotel, Point Clear, Ala.

May 15-16, 1961 — 41st Annual Meeting, Society of American Military Engineers, Washington, D.C.

May 16-18, 1961—1961 Spring Conferences, Building Research Institute, Shoreham Hotel, Washington, D.C.

June 4-8, 1961—Annual Meeting, Prestressed Concrete Institute, Denver-Hilton Hotel, Denver, Colo.

June 25-30, 1961—64th Annual Meeting, American Society for Testing Materials, Chalfonte-Haddon Hall, Atlantic City, N. J.

July 3-6, 1961—RILEM Symposium on Durability of Concrete, Prague, Czechoslovakia

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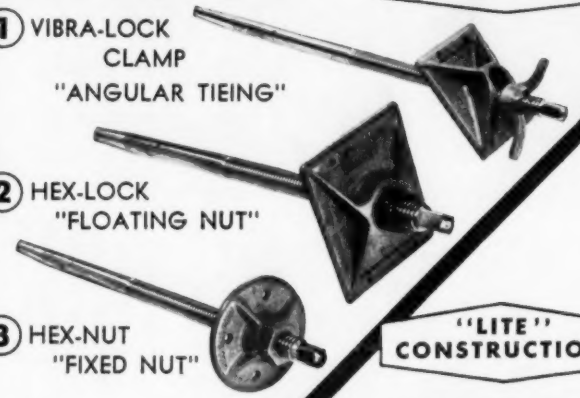

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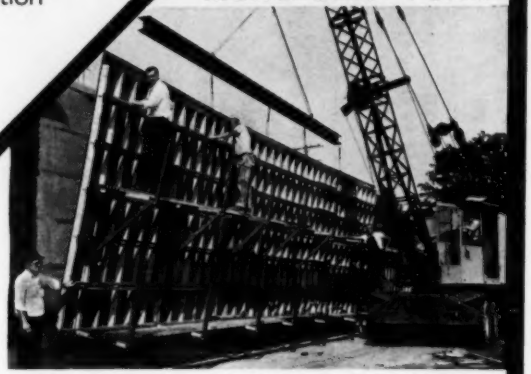

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Austin appoints Shirk to research post

Charles A. Shirk has been appointed manager of the Research Division of The Austin Co., international engineering and construction firm with headquarters in Cleveland. He will direct and coordi..ate staff services and operations for the Research Division headed by A. T. Waidelich, vice-president for research and engineering.

Mr. Shirk joined the organization 13 years ago as a structural engineer in the company's Chicago District. He was in charge of the structural design of the 25-million-dollar Automatic Electric Co. plant at Northlake, Ill. In 1957 he was named assistant district engineer in the Chicago District.

Mr. Shirk is a civil engineering graduate of the University of Notre Dame. He is a member of ACI, the National Society of Professional Engineers and the American Welding Society. He holds engineering registrations in 18 states.

AIME speaker cites improvement in cement materials handling

Alfred A. Roetzer, general manager, packaging services, St. Regis Paper Co., was one of the featured speakers at the national convention of the American Institute of Mining, Metallurgical, and Petroleum Engineers, in St. Louis. Mr. Roetzer spoke on improvements under way in materials handling and transportation in the cement industry's packaging methods, outlining a program now in process for more economical operation. Automatic checkweighing and automatic application of the empty valve bags to the packing machine are among new developments.

Mr. Roetzer said that the following production economies are beginning to shape up: Warehousing of filled bags permits longer packing runs of each type of cement with a minimum of down time. More intensive use of packaging equipment has resulted in the need for fewer packing and loading crews. Automation of the various functions means fewer and simpler job requirements, less fatigue. Mechanized fork truck handling of unit loads of 7 to 10 bbl of cement results in less operator fatigue and usually in lower labor cost requirements. Trucks can be loaded at odd hours by single fork truck operator without the need of calling out an entire packing and loading crew.

Additional benefits from the program develop in such areas as inventory control, bag loading, count, bag construction, bag cleanliness, less spill and breakage, cleaner packages, better weight accuracy, fewer customer complaints, shorter truck loading and unloading time, and the minimizing of safety hazards throughout the entire circuit.

Errata

The following correction should be made in "Shear Strength of Restrained Concrete Beams Without Web Reinforcement," by John E. Bower and Ivan M. Viest, which appeared in the July, 1960, ACI JOURNAL.

p. 77—heading for the fifth column of Table 1 should read: $\frac{P_2}{P_1}$.

* * * *

The following correction should be made in "Investigation of Compressive Strength of Molded Cylinders and Drilled Cores of Concrete," by Bryant Mather and William O. Tynes, which appeared in the January, 1961, JOURNAL.

p. 773 — change "$\sigma \cdot \nu = \sigma_b$" to "$\sigma \cdot y = \sigma_b$."

* * * *

The following correction should be made in "Rheological Behavior of Hardened Cement Paste under Low Stresses," by Joseph Glucklich and Ori Ishai, which appeared in the February, 1961, JOURNAL.

p. 949—in the third paragraph change Fig. 7 to Fig. 6.

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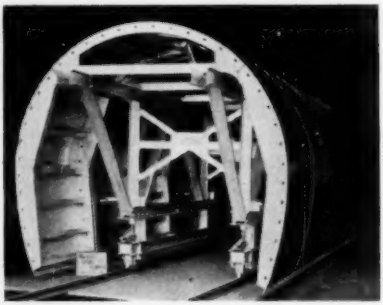

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Annual report
Continued from p. 4

with ACI Committee 622, Formwork, in the preparation of the ACI manual on formwork. Target date for completion of this work is fall 1961. Preliminary meetings have been held with the chairman of the committee; valuable assistance has also been given by Paul Barton, Birmingham, Mich., a newly appointed member of the committee.

Two bibliographies, "Evaluation of Strength Tests on Concrete" and "Fatigue of Concrete" were published during the year and added to the bibliography series. The Committee 326 bibliography on shear and diagonal tension is still in the hands of the committee.

Chapter activities

The Board of Direction authorized formation of a fourth chapter to include the state of Michigan plus Lucas County, Ohio. This group of ACI members had been meeting in Detroit monthly for more than a year to determine the continuing need and desirability of having chapter meetings. The earlier three chapters remained active and were instrumental in a number of educational programs in Oklahoma, southern California, and northern California. A project of the latter chapter is to provide libraries and universities and colleges in its locality with complimentary copies of the 55-Year Index.

Meetings have been held with a representative of the Hawaiian group of ACI members and it is expected that shortly an application will be received from that group to form an ACI chapter in that state. Expressions of interest, which will be followed up, have been received from ACI groups in Japan, Seattle, Houston, Washington, D. C., and Princeton, N. J.

Staff Aid

As soon as a new staff member, being sought for the Editorial Department, is obtained, the Assistant Secre-

tary will be free to do more field work and correspondence with groups of ACI members wishing to form ACI chapters.

Conventions

To aid the Technical Activities Committee in planning and carrying out convention programs, a questionnaire was distributed at the 1961 convention to determine member needs and desires in the way of technical papers and other features of the program. It is hoped that the results of these questionnaires will aid in directing programs into the most popular channels.

Advertising

Advertising revenue is a small but important item of ACI income. In 1960, net from advertising increased slightly as shown in Table 3.

Staff

A change of great potential importance in Institute operations during 1960 was the reorganization of the staff. The Board approved the creation of the new position of Assistant Secretary with principal duties the development of Institute chapters and close liaison with the TAC Committee on Monographs. Additional duties are handling the details of conventions and regional meetings. Robert E. Wilde was promoted from managing editor to the new position. His place as managing editor was taken by Robert G. Wiedyke and at the end of the year a search was still on for a replacement for Wiedyke as associate editor.

A consolidation of membership activities with the Advertising Department was effected with Thomas C. Akas being given the title Manager of Membership Department in addition to that of advertising manager. Under his supervision a smooth transition in transfer of membership records from Datamation, Inc. to Statistical Analysts, Inc., both in the Detroit area, was effected.

New employees joining the staff during 1960 were George D. Nasser, assistant editor; Mrs. Violet Rafferty and Mrs. Paula Schmalzriedt, Membership Department; and Miss Marilynn Johnson, Secretarial Department. Three employees, including Mr. Richard Mehoke, assistant editor; Mrs. Evelyn Schlitz, membership secretary; and Mrs. Russell, Secretarial, left the staff during the year.

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Merrill assumes presidency of Durox of Minnesota

Kenneth F. Merrill, formerly vice-president and treasurer of Durox of Minnesota, Inc., St. Paul, has been elected president and treasurer of the company. He is also president of Swedish - American Industries, Inc., United States licensor for the Durox cellular concrete building materials process.

Philip C. Bettenburg, formerly president, was elected to the newly created post of chairman of the board.

William Mack Angas

Admiral William Mack Angas USN (Ret.), chairman and professor of civil engineering, Princeton University, Princeton, N. J., died Dec. 12, 1960.

Admiral Angas was born in Great Burdon, County of Durham, England, July 5, 1892, coming to the United States in 1895. He graduated from the Massachusetts Institute of Technology, Cambridge, in 1917.

Accepting an appointment as assistant civil engineer with the rank of Lieutenant (Junior grade) on Dec. 27, 1917, he began a long and illustrious naval career, culminating some 33 years later with his retirement from active duty in the navy with the rank of vice-admiral. In 1950 he was named chairman of civil engineering at Princeton University.

Admiral Angas had received Letters of Commendation for work done on repairing Drydock No. 3 at the Navy Yard, Philadelphia, in 1921, and for work done for controlling a flood in San Diego River, Calif., in February, 1927. He was awarded the Victory Medal and the American Defense Service Medal.

He was the author of "Rivalry on the Atlantic," a history of the development of the commercial express steamship on the Atlantic Ocean. Many of his professional articles have been published in technical journals. In 1944 he was awarded the ACI Wason Medal for his paper, "Concrete Problems in the Construction of Graving Docks by the Tremie Method," which he co-authored with E. M. Shanley and John J. Erickson.

Eivind G. Elstad

Eivind G. Elstad, 58, died on December 7 after a long illness.

Born in Oslo, Norway, he came to this country in 1925 and became quickly known as a man of eminent engineering skill, highly esteemed by engineers, architects and builders who had occasion to work with him. He was responsible for many structures for which his firm acted as consultants, among which the Thule Air Force Base in Greenland, Tripler General Hospital in Hawaii, the Seagram Office Building in New York are just a few examples. He was a recognized authority on construction in arctic climates.

Mr. Elstad was an ACI member of long standing, a Fellow of the American Society of Civil Engineers, and a member of the New York Association of Consulting Engineers.

A. Sagorsky

A. Sagorsky, chemist and managing director, Salt River Cement Works (Pty.) Ltd., Capetown, South Africa, recently died. Mr. Sagorsky had been a member of ACI since 1930.

Kenneth S. Iwata

Kenneth S. Iwata, principal of the firm of Kenneth S. Iwata and Associates, Inc., Los Angeles, drowned in a fishing accident in Sacramento River on Nov. 13, 1960.

Mr. Iwata, a graduate of Washington University, St. Louis, Mo., attended the University of South California, Berkeley, receiving MS degrees in civil engineering and mechanical engineerin. He was a lecturer at USC and was working toward a Doctor's degree.

Mr. Iwata had worked for various engineering firms in Los Angeles since 1947 before entering private practice in 1956. He had served as structural engineer for the Santa Barbara Five-Point Shopping Center; UCLA library; Los Angeles Federal Savings and Loan in Van Nuys; University of California at Santa Barbara; Flamingo Hotel addition in Las Vegas; and numerous other structures in Southern California, Nevada, and Arizona.

He had been an ACI member since 1957 and was active in the ACI Southern California Chapter.

Honor Roll

Jan. 1-Jan. 31, 1961

J. W. Connell proposed 9 new members this month putting him in first place at the beginning of the new Honor Roll year.

New ACI members stem from various sources, however the greatest number are prompted to membership by persons already on the membership rolls of the Institute.

Let's keep the membership trend spiraling. Add your name to the Honor Roll next month.

J. W. Connell	9
K. J. Cavanagh	6
S. Hobbs	5
P. W. Abeles	3
J. R. Fiorey	3
P. H. Greenfield	3
A. T. Hersey	3
Joe W. Kelly	3
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L. R. Lauer	3
G. S. Maynard	3
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M. Stein	3
P. B. Tenchavez	3
T. W. Thomas	3
E. S. Vieser	3
V. S. Winkel	3
H. Yashiro	3
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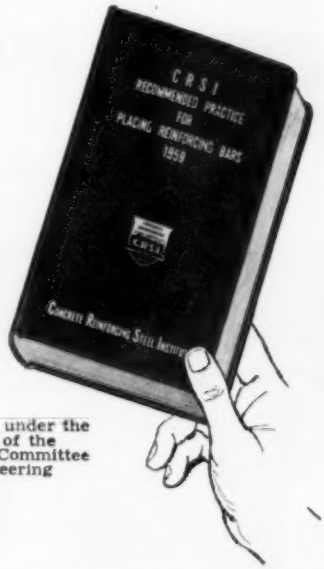

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New Members

The Board of Direction approved applications in the following categories: 66 Individual, 1 Corporation, 16 Junior, and 18 Student, making a total of 101 new members. Considering losses due to deaths, resignations, and nonpayment of dues, the total membership now stands at 10,348.

INDIVIDUAL

ALSINA, EDUARDO LUIS, Monterey, Calif. (Desr., Carter & Slattery)

ANSLEY, R. H., Vancouver, B. C., Canada (Vice-Pres., Commonwealth Constr. Co., Ltd.)

BALDOCK, BRIAN C., Melbourne, Victoria, Australia (Struct. Engr., Clive Steele Assocs., Cons. Engrs.)

BHAGAT, VITHALBHAI TRIKAMBHAI, Chicago, Ill. (Struct. Engr., Procon, Inc.)

CAPLAN, LOUIS, Johannesburg, South Africa (Mng. Dir., A. S. Joffe & Co. (Pty.) Ltd.)

CHENG-LU-I, Hayward, Calif. (C. E., Rogers Engrg. Co.)

CHEPAITIS, S. P., Hudson, N. Y. (Plant Chf. Chem., Lone Star Cemt. Corp.)

CRAVEN, A. R., Seattle, Wash. (Plant Chf. Chem., Lone Star Cemt. Corp.)

CRNEKOVIC, STJEPAN, Chicago, Ill. (Struct. Desr., A. Benesch & Co.)

CUTCLIFFE, J. Lloyd, Cambridge, Mass. (Instr., C. E., M. I. T.)

DANCE, IAN M., Melbourne, Victoria, Australia (Struct. Engr., Clive Steele Assocs., Cons. Engrs.)

DAVID, THOMAS C., Alexandria, La. (Owner, Pan American Engrs.)

DAVIS, JOE L., III, Shreveport, La. (Chf. Engr., Indust. Steel Prods. Co., Inc.)

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GALE, ERNEST L., Melbourne, Victoria, Australia (Struct. Engr., Clive Steele Assocs., Cons. Engrs.)

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Many of you may have received this coupon already. If not, watch for it. By returning the coupon intact (your addressed mailing label is included) **by April 14, 1961,** a copy will be reserved for you. Because of increased printing costs, only enough to meet membership requests will be printed.

Accompanying this special coupon is a form for submitting the names and addresses of prospective ACI members. Even if you don't want a copy of the Directory, we hope you'll use this second form to send us the names of several member prospects. You'll receive full credit in the JOURNAL Honor Roll for the members joining as a result of your recommendation.

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SETTERINGTON, GEORGE, Leamington, Ont., Canada (Univ. of Western Ontario)
SINGHAL, AVINASH C., Cambridge, Mass. (M. I. T.)

Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

AED 1960-61 INDUSTRY DIRECTORY—Completely revised and re-edited, the 400-page edition represents a comprehensive reference source and buyers guide. $20 per copy.—Associated Equipment Distributors, 30 East Cedar St., Chicago, Ill.

CALCIUM CHLORIDE FOR CONCRETE CONSTRUCTION (CB-4)—Information sheet prepared for ready-mixed concrete operators, contractors, and engineers who wish to accelerate the set of concrete, especially when outside temperatures fall below 70 F.—Calcium Chloride Institute, Ring Building, Washington 6, D. C.

INDUSTRIAL BELT BUCKET ELEVATORS (Bulletin No. 174)—New 22-page booklet contains engineering data on various types of bucket elevators and recommendations for the grades of belting best suited for elevating materials of different weight, abrasiveness, temperatures, and other characteristics. —Hewitt-Robins Inc., 666 Glenbrook Rd., Stamford, Conn.

QUALITY PRESTRESSED CONCRETE (MBR No. 13)—The unlimited applications for high quality prestressed concrete are described in 16 studies of outstanding prestressed concrete projects in this 20-page publication. A prestressed lift slab, 120 ft long prestressed bridge girders, and a 2 million gal. prestressed water tank are some of the outstanding projects as both pre-tensioned and post-tensioned projects are studied. The role played by Pozzolith in achieving the high concrete qualities required for prestressed work is also treated.—The Master Builders Co., Cleveland 18, Ohio

HOW TO TURN CONCRETE INTO GOLD—Combination catalog and instruction manual for producers of ornamental concrete products lists and illustrates more than 280 different aluminum alloy molds for production of block, columns, flower boxes, statuary, stepping stones, posts, and similar items. The 88-page book offers the novice a simplified introduction to concrete technology and covers preparation of the molds, conventional casting or alternate semidry tamping, removal of the molds, and curing. Also discussed are admixtures, coloring, painting, and development of exposed aggregate surfaces as well as reinforcement for some of the more intricate pieces of statuary. Suggestions for promotion and sales of ornamental concrete are offered. Price $2.00—Concrete Machinery Co., Inc., Drawer 60, Hickory, N. C.

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